U.S. DEPARTMENT OF COMMERCE
National Technical Information Service

PB-265 634

Proceedings United States/Japan Conference on Sewage Treatment Technology (4th), Held at Cincinnati, Ohio on Oct 23-24, 1975, & Washington, D.C. on Oct 28-29, 1975

Environmental Protection Agency, Cincinnati, Ohio

Oct 76

A-600/9-76-023

PB 265 634

October 23-24 October 28-29
Cincinnati, Ohio Washington D.C.

Proceedings

4th
United States/Japan
Conference
Sewage Treatment
Technology

1975

REPRODUCED BY
NATIONAL TECHNICAL
INFORMATION SERVICE
U. S. DEPARTMENT OF COMMERCE
SPRINGFIELD, VA. 22161

U.S. Environmental Protection Agency

TECHNICAL REPORT DATA

(Please read Instructions on the reverse before completing)

1. REPORT NO. EPA-600/9-76-023	2.	3. RECIPIENT'S ACCESSION NO.
4. TITLE AND SUBTITLE PROCEEDINGS, FOURTH UNITED STATES/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY		5. REPORT DATE October 1976 (issuing date) 6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S)		8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS U. S. Environmental Protection Agency *		10. PROGRAM ELEMENT NO. IRW 103 11. CONTRACT/GRANT NO.
12. SPONSORING AGENCY NAME AND ADDRESS U. S. Environmental Protection Agency * - Cin., OH		13. TYPE OF REPORT AND PERIOD COVERED Proceedings - Oct. 23-27, 197 14. SPONSORING AGENCY CODE EPA/600/14

15. SUPPLEMENTARY NOTES *Symposium sponsored by Office of Int'l Activities, Office of Water & Hazardous Materials (Wash.,D.C. 20460), & Office of Research & Development (Cinti.OH 45268 & Wash.,D.C. 20460). This volume prepared & published by Office of Research & Development, Cincinnati, Ohio 45268.

16. ABSTRACT

As part of joint interests in environmental matters between the United States and Japan a Conference on Sewage Treatment Technology is held at intervals of about 18 months. This publication contains papers from the Japanese group and from the American side that were presented at the Fourth Conference. Subject matter covered includes sludge treatment and disposal, automation and instrumentation, advanced waste treatment, planning and management of wastewaters, storm and combined overflows, and industrial waste treatment. The publication is unique in that it presents, in English, comprehensive information on wastewater treatment and research being conducted in Japan.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Sewage treatment, Sludge disposal, Industrial waste treatment, Waste disposal	Water pollution control	13B

18. DISTRIBUTION STATEMENT To Public	19. SECURITY CLASS *(This Report)* unclassified	21. NO. OF PAGES 681
	20. SECURITY CLASS *(This page)* unclassified	22. PRICE PC A99 / MF A01

EPA Form 2220-1 (9-73)

☆USGPO: 1977 — 757-056/5496 Region 5-11

EPA-600/9-76-023
October 1976

PROCEEDINGS

FOURTH UNITED STATES/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

Cincinnati, Ohio: October 23-24, 1975
Washington, D.C.: October 28-29, 1975

Office of International Activities
Office of Water and Hazardous Materials
Washington, D.C. 20460

Office of Research and Development
Washington, D.C. 20460
Cincinnati, Ohio 45268

U.S. ENVIRONMENTAL PROTECTION AGENCY
OFFICE OF RESEARCH AND DEVELOPMENT
CINCINNATI, OHIO 45268

ia

DISCLAIMER

These Proceedings have been reviewed by the U.S. Environmental Protection Agency and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the U.S. Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

FOREWORD

The industry and technical ability of the Japanese and American people have brought prosperity to our two nations but have also created grave strains on the environment. Both nations are conscious of the need to control pollution and protect the environment.

US-Japan environmental cooperation has been going on for several years. The fruits of our mutual labors are becoming visible. The United States and Japan have now completed the fourth in a series of Joint Conferences on Sewage Treatment Technology. Both countries have profited from this exchange. These Proceedings, representing the most up-to-date information in several areas of wastewater treatment, will be welcomed for their contribution to our knowledge in this field.

Russell E. Train

Russell E. Train
Administrator

Washington, D.C.
July, 1976

CONTENTS

FOREWORD iii

JAPANESE DELEGATION vi

U. S. - CINCINNATI DELEGATION vii

U. S. - WASHINGTON DELEGATION viii

JOINT COMMUNIQUE 1

JAPANESE PAPERS 2
    1 THROUGH 8

EPA - CINCINNATI PAPERS 503

EPA - WASHINGTON PAPERS 628

Preceding page blank

JAPANESE DELEGATION

DR. TAKESHI KUBO
Head of Delegation, Executive Director, Japan Sewage Works Agency

TOHRU HAYASHI
Head, Water Quality Control Division, Water Quality Bureau, Environmental Agency

DR. MAMORU-KASHIWAYA
Head, Water Quality Control Division, Public Works Research Institute, Ministry of Construction

KEN MURAKAMI
Chief, Water Quality Section, Water Quality Control Division, Public Works Research Institute, Ministry of Construction

DR. AKINORI SUGIKI
Head, Research and Technology Development Division, Japan Sewage Works Agency

MASAYUKI SATO
Director, Sewage Works Bureau, Yokohama City Office

SEIICHI YASUDA
Director, Sewage Works Bureau, Kyoto City Office

SATORU TOHYAMA
Head of Sewage Works Division, Dept. of Sewerage & Sewage Purification, Ministry of Construction

UNITED STATES/CINCINNATI DELEGATION

FRANCIS M. MIDDLETON
General Chairman of the Conference and
Head of the Cincinnati U.S. Delegation;
(Acting) Senior Science Advisor, MERL

LOUIS W. LEFKE
(Acting) Director, MERL

EDWIN F. BARTH
Chief, Biological Treatment Section,
TPDB, WRD, MERL

RICHARD C. BRENNER
Sanitary Engineer, Biological Treatment
Process Branch, WRD, MERL

JOHN CIANCIA
Sub-Program Chief, Industrial Treatment
and Control, Industrial Environmental
Research Laboratory (Edison, New Jersey)

JESSE M. COHEN
Chief, Physical-Chemical Treatment
Section, TPDB, WRD, MERL

JOHN J. CONVERY
(Acting) Director, WRD, MERL

JOHN N. ENGLISH
Sanitary Engineer, Municipal Treatment
and Reuse Section, SEEB, WRD, MERL

DR. JOSEPH B. FARRELL
Chief, Ultimate Disposal Section, TPDB,
WRD, MERL

RICHARD I. FIELD
Chief, Storm and Combined Sewer Section,
SEEB, WRD, MERL (Edison, New Jersey)

ROBERT A. OLEXSEY
Mechanical Engineer, Ultimate Disposal
Section, TPDB, WRD, MERL

JOSEPH F. ROESLER
Sanitary Engineer, Pilot and Field Evaluation Section, Technology Development
and Support Branch, WRD, MERL

B. VINCENT SALOTTO
Research Chemist, Ultimate Disposal
Section, TPDB, WRD, MERL

DR. JAMES E. SMITH
Sanitary Engineer, Ultimate Disposal
Section, TPDB, WRD, MERL

JAMES J. WESTRICK
Sanitary Engineer, Physical-Chemical
Treatment Section, TPDB, WRD, MERL

MERL - Municipal Environmental Research Laboratory
WRD - Wastewater Research Division
TPDB - Treatment Process Development Branch
SEEB - Systems and Engineering Evaluation Branch

UNITED STATES/WASHINGTON DELEGATION

FRANCIS M. MIDDLETON
General Chairman of the Conference; (Acting) Senior Science Advisor, MERL (Cincinnati, Ohio)

JOHN T. RHETT
Head of Washington U.S. Delegation; Deputy Assistant Administrator for Water Program Operations, OWHM

CHARLES H. SUTFIN
Deputy Head of Washington U.S. Delegation; Deputy Director, Municipal Construction Division, Water Program Operations, OWHM

ROBERT A. CANHAM
Executive Secretary, Water Pollution Control Federation

FRANCIS J. CONDON
Sanitary Engineer, Community Sources Staff, Waste Management Division, Air, Land, and Water Use, ORD

FITZHUGH GREEN
Associate Administrator for International Activities

ERNEST P. HALL
Deputy Director, Effluent Guidelines Division, Water Planning and Standards, OWHM

JOE G. MOORE, JR.
Program Director, National Commission on Water Quality

WALTER S. GROSZYK
Deputy Director, Water Planning Division, Water Planning and Standards, OWHM

WILLIAM A. ROSENKRANZ
(Acting) Director, Waste Management Division, Air, Land, and Water Use, ORD

ROBERT B. SCHAFFER
Director, Permits Division, Water Enforcement, Office of Enforcement

DR. WILSON K. TALLEY
Assistant Administrator for Research and Development

CALVIN C. TAYLOR
Economic Analyst, Interagency Liaison Office, Region IV (Atlanta, Georgia)

ADDITIONAL PARTICIPANT

KIRK MACONAUGHEY
Japanese Coordinator, Office of International Activities

OWHM - Office of Water and Hazardous Materials
ORD - Office of Research and Development
MERL - Municipal Environmental Research Laboratory

DR. TAKESHI KUBO, JAPANESE TEAM LEADER
OPENS THE CONFERENCE AT EPA IN CINCINNATI, OHIO

UNITED STATES OF AMERICA
NTAL PROTECTION AGE
ONMENTAL RESEARCH CENTER

JAPANESE TEAM VISITS THE NEW EPA
ENVIRONMENTAL RESEARCH CENTER, CINCINNATI, OHIO

JAPANESE VISIT ROCKY RIVER, OHIO
PHYSICAL-CHEMICAL TREATMENT PLANT

JAPANESE VISIT CONSTRUCTION SITE
OF PHYSICAL-CHEMICAL TREATMENT PLANT
NIAGARA FALLS, NEW YORK

JOE G. MOORE (FAR LEFT) ADDRESSES JOINT CONFERENCE - WASHINGTON, D.C.

Joint Communique

Fourth U.S./Japan Conference on Sewage Treatment Technology

Washington, D. C.

October 29, 1975

1. The Fourth United States/Japan Conference on Sewage Treatment Technology was held in Cincinnati, Ohio and Washington, D. C., from October 23-29, 1975.

2. The Japanese delegation, headed by Dr. Takeshi Kubo, Executive Director, Japan Sewage Works Agency, was composed of six National Government officials and two local Government officials.

3. Mr. Frank M. Middleton, Senior Science Advisor, U. S. Environmental Protection Agency, Cincinnati, Ohio, was General Chairman of the Conference. Mr. John T. Rhett, Deputy Assistant Administrator for Water Program Operations was Head of the Washington delegation. Mr. Charles H. Sutfin, Deputy Director, Municipal Construction Division, was Deputy Head of the Washington delegation. In addition to EPA officials, Conference delegates included Joe G. Moore, Jr., Program Director, National Commission on Water Quality, and Robert A. Canham, Executive Secretary, Water Pollution Control Federation.

4. Prior to the Conference the Japanese delegates visited advanced waste treatment facilities in Orange County, California, and Escondido, California. Physical chemical treatment plants were seen in Rocky River, Ohio, Cleveland, Ohio, and Niagara Falls, New York. The pure oxygen treatment plant in Detroit, Michigan was visited. In Washington, D. C., the EPA Blue Plains Sewage Treatment Pilot Plant was visited and a tour of the Piscataway tertiary treatment plant was made.

5. Principal topics of the Fourth Conference in Cincinnati were status of pure oxygen use, sludge handling and disposal by heat treatment, incineration and land disposal, urban storm water technology, automation and instrumentation, use of activated carbon, filtration, phosphorus removal, industrial waste treatment progress, reuse and disinfection. In Washington the U. S. side discussed aspects of Public Law 92-500 including planning, urban runoff, permits, pretreatment, construction grants status and the tentative conclusions of the National Commission on Water Quality. The Japanese side discussed environmental improvement in Japan, comprehensive planning, combined sewer overflow technology, pretreatment and case histories of industrial waste treatment. Vigorous discussions followed the conference presentations.

6. Recent personnel exchanges include a month-long visit to Japan by Dr. James E. Smith and Mr. Dolloff F. Bishop of the EPA Taft Center, Cincinnati, Ohio. Dr. K. Inaba, Japan Sewage Works Agency, is spending six months in the United States to study storm sewer flows and urban runoff problems.

7. In the research area it was agreed to enter into joint projects in areas of municipal sludge disposal technology, agricultural use of sludge, nitrogen control technology, instrumentation and automation of wastewater treatment plants and other interests to both countries. Researchers on both sides will exchange information on a regular basis. Full reports will be given at future conferences.

8. It was proposed by the Japanese side that the Fifth U. S./Japan Conference should be held in Japan, about May 1977.

9. A Proceedings of the Conference will be printed in English and Japanese.

Fourth US/JAPAN Conference
on
Sewage Treatment Technology
Paper No. 1

SLUDGE TREATMENT AND DISPOSAL

October 24, 1975
Cincinnati, Ohio

Ministry of Construction
Japanese Government

SLUDGE TREATMENT AND DISPOSAL

1. **Sludge Handling Practice in Japan** 4
   A. Sugiki, Japan Sewage Works Agency
2. **Studies on Performance for Sewage Sludge Dewatering Process** 10
   A. Sugiki, Japan Sewage Works Agency
3. **Sewage Sludge Treatment and Disposal as Practiced in Yokohama City – "Reclamation to Agricultural Land and Green Field" Program –** 25
   M. Sato, Yokohama City
4. **The Treatment and Disposal of Sludge at Toba Treatment Plant** 33
   S. Yasuda, Kyoto City
5. **Sludge Production and Solid Loading Balance in the Nishiyama STP, Nagoya** 46
   T. Annaka and M. Kashiwaya, PWRI, Ministry of Construction

CHAPTER 1. SLUDGE HANDLING PRACTICE IN JAPAN

1.1 Introduction 5
1.2 Dewatering and Incineration of Sewage Sludge 6
1.3 Research and Technological Development 8

1.1 INTRODUCTION

The number of publicly owned sewage treatment plants in operation in Japan amounts to 169 as of 1973. The sludge by weight produced out of those plants is extremely massive. The volume by weight per capita of sludge produced is estimated in Japan on the basis of Suspended Solid, SS, content in the incoming sewage.

Since the grams per capita of SS in sewage is, at present, found to stand at 40 grams and BOD_5 at 44 grams, the sludge volume by weight produced comes up to 36 grams on condition that the removal rate of SS is 80 percent by secondary treatment process. On the other hand, according to a survey conducted by the Japan Sewage Works Agency, 20 liters of sludge with the solid contents having 2 percent on an average are being produced per capita a day, indicating a value almost close to the calculated above. It is known, however, that at some treatment plants where the activated sludge process is adopted the sludge volume produced exceeds the value mentioned above approximately by 10 percent.

As for the sludge generated in the treatment process, it is necessary to study the sludge in preliminary sedimention basin, the excess sludge originating from activated sludge, the suspended solids existing in recycle flow that grows out of a sludge treatment system and the material balance of sludge in a treatment system. Recently, the necessity of surveys of this kind has been keenly recognized and results of such attempts are partly described in this report from the City of Kyoto and a report on the Nishiyama Treatment Plant, Nagoya City studied by the Public Works Research Institute, Ministry of Construction, both to be referred to in this paper.

According to the studies carried out by the Ministry of Construction in 1967, systems of sludge treatment and disposal can be summarized as follows. The most prevailing system in use at 169 treatment plants which had been then in operation was that of "thickening–unaerobic digestion–mechanical dewatering", accounting for 35 percents while 25 percents was for that of "unaerobic digestion–drying bed". The treatment plants where sludge was incinerated were only those 10 established in big cities.

As for disposal systems, the land-filling was adopted at 97 treatment plants and the sludge from 30 treatment plants are applied on agricultural land as fertilizer and/or soil conditioner, thus sludge having been returned to the earth at 80 percents of the total treatment plants.

A study made in 1973 shows that the treatment plants where the system of "unaerobic digestion–mechanical dewatering" was adopted numbered 77, accounting for 29 percents of all, while at 55 treatment plants (21 percents) raw sludge was directly dewatered by a machine. The number of plants where drying bed was installed was 24, indicating a decrease at the time of 1967.

In the case of small-scale treatment plants, an increase was noticed in the adoption of aerobic digestion. 34 treatment plants were equipped with incinerator, accounting for 13 percent of the total, and at 17 of them the system of "raw sludge dewatering–incineration" was adopted while that of "digestion–dewatering–incineration" was at the remaining 17.

Types of filtering / dewatering machine shown in the results of 1973 study are classified as follows:

–vacuum filter;	135 treatment plants	381 sets
–filter press;	14 treatment plants	39 sets
–centrifuge;	20 treatment plants	36 sets
Total	169 treatment plants	

Also types of incinerating / drying apparatus are as follows:

–vertical multiple-hearth incinerator;	32 treatment plants	39 units
–rotary kiln;	6 treatment plants	6 units
–fluidized bed incinerator;	2 treatment plants	2 units

The reason why the system of "unaerobic digestion–mechanical dewatering (drying bed)–land-filling" heretofore in use has gradually been switched over to that of "raw sludge dewatering–incineration–land-filling" is because sludge became necessary to be treated efficiently with the increase in sludge volume produced, as well as because the necessity arose to reduce as much as possible the volume of sludge to be disposed, as it became difficult to find available land for sludge disposal. Some other reasons why the latter system became more in use are considered to be because

- land required for digestion and drying bed turned out to be extremely difficult;
- the former system requires longer sludge detention time and more labor for sludge treatment;
- the operation of the former is rather complicated and necessitates skill labor.

However, as the methane gas produced from unaerobic digestion has been put to use or sludge has been utilized for an agricultural soil conditioner since the oil crisis from the viewpoint of resources and energy saving, the need to reevaluate unaerobic digestion from the sanitary engineering aspect started to be recognized increasingly.

1.2 DEWATERING AND INCINERATION OF SEWAGE SLUDGE

As mentioned above, in respect to types of sludge dewatering machine a great number of treatment plants were found to have adopted vacuum filters. 4 out of 16 filter presses were in use for dewater sludge after the heat treatment. Ferric choloride ($FeCl_3$) and lime ($Ca(OH)_2$) were applied for conditioning to dewatering. Moisture content was 70 to 75 percents in vacuum filter and 55 to 60 percents in filter press. On the other hand, sludge treated the heat treatment was able to be dewatered down to the moisture content of 35 to 40%. According to a research made by the Agency last year on the actual conditions of dewatering devices in use at medium- and small-scale treatment plants, the volume of chemicals dosed fairly exceeded the designed value. This seems mainly to have aimed at surer operation of dewater, but its cause is now under investigation in detail.

Centrifuges were once employed for the physicochemical treatment of nightsoil and also employed for sewage sludge dewatering at some of the sewage treatment plants. But this type had not been adopted so widely due to the poor solids recovery and the problems of vibration and noise. Nevertheless, this type of machine is suitable for medium- and small-scale treatment plants on account of such merits that

- it can be compact in size after being kept in a small container for noise and odor control.

- it can run automatically in continiously and its maintenance is easy;
- it requires auxiliary facilities less.

This may also be reasoned out from the fact that in those cities where this type is adopted many of the treatment plants are given the sludge treatment capacity of more than 4 cubic meters but less than 100 cubic meters. In this type, the moisture content can be reduced to 80% to 85% by adding polyelectrolytes (or polymer) in the centrifugal dewatering, and since usual lime and ferric chloride are not applied, the volume of ash necessarily lessens when incinerating sludge. These merits of centrifuge in the operation and maintenance coupled with a merit in the incineration have accordingly led big cities to consider the possible adoption of this type.

A new series of this type of machine has begun to be introduced, with its number of revolutions reduced to 1,200 to 1,900 revolutions per minute and vibration, noise and durability are improved.

As for incinerators, the reason why many of them are of vertical multiple-hearth type is because its function has been strengthened so much, after years of technological improvement, as to be able to treat sewage sludge almost without any difficulty. Assuming that the moisture content of sludge is 70%, the capacity of incinerator is found to be 5 to 250 tons per day in the case of multiple-hearth type, 4 to 60 tons per day in rotary kiln and 5 to 20 tons per day in fluidized bed type. Rotary kiln consumes much fuel and seems to be considered inadequate for the adoption from the standpoint of energy conservation. Fluidized bed incinerator is suitable for medium- and small-scale plants, since the structure is simplified and the intermittent operation is economically feasible due to its high heat capacity. Such defects, however, are pointed out that the volume produced of NOx and powdered dust is considerable, and the power cost of blower is high as it requires high-pressure air. Yet, in order to overcome these defects such improvements have been made as to increase thermal efficiency by providing a spiral flow inside the funance and so on. As a result, more this type of incinerator are coming to adopt it.

Stack effluent treatment is a matter to be taken into account inevitably when incineration is brought into focus. As the restrictions on air pollution have gradually strengther, the treatment of waste gas from sludge incinerators accordingly came to require more complicated-one. Soot and dust, gases of SOx, HCl, Cl_2 and others in exhaust gas are considered harmful, and to cope with them scrubbers or washing with alkali and so forth have been employed. Recently, however, a wet-type electric precipitator and / or a after burning apparatus have been obliged to be added further in some areas. In the regions where a strict standard is enforced, the cost of equipment for waste gas treatment has gone so high as to exceed 50 percents of the cost of total incineration facilities. As a result, under study are new methods, for example, of heat treatment by which sludge is dewatered without dosing much of the chemicals but treatment of cooking liquors, would be required to develop in future.

On account of the fact that considerable energy is consumed in the incineration of sludge, the introduction of methods based on a new idea of incinerating with the least possible energy is now under study. For instance, the first machine of the so-called CG Process in which sludge is evaporated after being mixed with oil is selected for sewage treatment plant of Fukuchiyama City, and a Lucas-type incinerator in

Maebashi City, both being under construction at present. The Agency has been studying the performances of sludge dewatering since last fiscal year, of which result will be partly reported to this conference, and starting this fiscal year another study on the performances of incinerators and waste gas treatment facilities was decided to be carried out. This program is aimed to produce basic data for the preparing of rational design criteria, on the standpoint of systematic approach.

As for new-type facilities for sludge treatment set up in the cities of Fukuchiyama and Maebashi, it is planned by the Agency to analyse their process performance and to evaluate them, upon coming into actual operation.

1.3 RESEARCH AND TECHNOLOGICAL DEVELOPMENT

With the increase in the volume of sludge to be disposed, the Ministry of Construction, Japanese government, decided to augment to a great extent the budget for study and research on sludge treatment and disposal starting this fiscal year.

Sludge has been utilized for an agricultural fertilizer or a soil conditioner since long time ago. The Japan Society of Civil Engineers has been studying sewage sludge application on agricultural land since 1969.

The volume of sludge to be applied depends upon what plant to be cultured, but in the case of paddy rice the upper limit of application was considered to be 100 kilograms per ha., and the desirable rate of an application to vegetables and fruit trees, etc. was found to be more or less 250 kilograms per ha. It is of necessity, however, to make clear how soils and plants will be affected by heavy metals contained in sludge. The Japan Society of Civil Engineers studied this subject for two years, and starting this fiscal year the Agency is supposed to carry out an ecological study on the effect over soils and plants when sludge is applied continuously.

Yokohama City started a study on the use of sewage sludge on agricultural land in 1971 and constructed a drying facility based on the study results. That facility is now in practical use and further detail will be given in the chapter III.

Various attempts to recover the useful ingredients in sludge have been made so far, and the one to utilize an available gas after drying sludge up by distillation began to be studied also in our county.

The supernatant originating from sludge digestion and heat treatment used to be sent back to a main plant and treated there. As far as the heat treatment supernatant is concerned, it is known that in Sapporo City and so forth a considerable portion can be removed through biological treatment. However, this kind of supernatnat contains a sizable amount of phosphorus and nitrogen aside from organic matters, and when the supernatant is sent back to a main plant those phosphorus and nitrogen are to be discharged fairly much into the treated water. Places where the supernatant seems necessary to be treated separately from the standpoints of waste water treatment and eutrophication measures are increasing in number. The Agency has started a study and research on this problem since the beginning of this fiscal year.

On the other hand, the Public Works Research Institute has begun to study how to recover organic matters from the supernatant to utilize for carbonic resources for biological denitrification.

The development of treatment and disposal method of tertiary-treatment sludge is one of the important subjects to be studied more thoroughly in future. The Public Works Research Institute has already started a basic research on the recovery of tertiary-treatment sludge, etc., and also the Agency is conducting a study and research on the dewatering of sludge from a pilot plant at the Lake of Biwa, Shiga Prefecture. This paper covers hereinafter the following contents.

II. Studies on Performance for Sewage Sludge Dewatering Processes

III. Sewage Sludge Treatment and Disposal in Yokohama–Agricultural Land Application.

IV. Sludge Treatment and Disposal in Toba Sewage Treatment Plant, in Kyoto

V. Material Balance of Solids in Nishiyama Sewage Treatment Plant in Nagoya City

CHAPTER 2. STUDIES ON PERFORMANCE FOR SEWAGE SLUDGE DEWATERING PROCESSES

2.1 Purpose 11

2.2 Statistics 11
  2.2.1 Sewage Plant and Dewatering Equipment 11
  2.2.2 Thickening 11
  2.2.3 Sludge Cake and Filtrate 12
  2.2.4 Conditioning 12
  2.2.5 Performance 12

2.1 PURPOSE

At present, throughout Japanese main lands, 426 sewage plants are under operation. Recently a activated sludge process becomes a standard means to cope with urban waste water. The process produces a large amount of sludge which requires another treatment. Therefore, almost all of the sewage plants have sludge dewatering equipments.

Although design parameters of the dewatering machines are provided in some manuals, performances of the machines are not certain. Once machines are set up, not only design personnels but also regulatory agencies lost interests in them. Operational experience and performance data are accumulated only locally. As a result, design technology of the machines does not improve but still remains under-development. A procedure to select the proper type which is most suitable for the local conditions is "the rule of a thumb". If his machine works properly, he is in bonanza. It is a little more than an art to determine his type.

Through the year of 1975, sewered population will exceed twenty percent. The rest 80 percent people are scheduled to be sewered within few years. Therefore Japan is an attractive market for sewage equipment manufacturers. Their advertisement is like a kaleidoscope. This adds another confusion to local personnels.

Japan Sewage Works Agency is responsible to provide municipalities with engineering consultations. We launched a research project aiming primarily to establish "a standard procedure to select sludge dewatering equipment and to make a specification of the dewatering machines required for satisfying the local conditions".

The first step approaching to this goal is to overview the present dewatering practices. Our staffs visited 170 sewage plants and collected operational data to build a statistics which will show a rough sketch of the sewage sludge dewatering practices in Japan.

2.2 STATISTICS

2.2.1 SEWAGE PLANT AND DEWATERING EQUIPMENT

The main process for waste water treatment and design capacity of the 170 plants surveyed are summarized in Table 2.1. 64% of the total 170 plants are designed on the basis of the standard activated sludge process and 28% are of the step aeration process. A majority of the small plants whose capacity is less than 5,000 m^3/day has the standard activated sludge process. A half of the middle class plants where capacity ranges from 20,000 to 100,000 m^3/day has the standard activated sludge and the rest half has the step aeration.

The continuous rotary vacuum filter is the most commonly used device for dewatering sewage sludge. This device is used widely in the small and large sewage plants. 80 percents of the 170 plants are equiped with this (Table 2.2).

2.2.2 THICKENING

Solids concentration in mixture of primary and secondary sludge ranges widely. Annual average values of the concentration are distributed from 0.5 percents to 3.0 percents. It is inefficient to feed the dewatering equipment. They are too thin and should be concentrated.

51 plants have only thickeners and 43 plants have primary thickeners, digesters, elutriation, tanks, and secondary thickeners. The digesters are found in 101 sewage plants. As a general rule the small sewage plants have only the thickener as a pretreatment equipment. The digesters are practiced in a half of the large sewage plants (Table 2.3).

A number of the centrifuge that follows the digestion tanks is large. The half number of the rotary vacuum filters is fed with digested sludge. All the filter press and the rest half vacuum filters are fed with mixture of raw primary and secondary sludge (Table 2.4).

4 percent solid concentration is a standard value for design of sludge dewatering equipment. However, the annual average of this value in 102 plants exceeds this standard value (Table 2.5).

No significant difference in solid concentration between digested and undigested sludge appears.

As compared with the other sludge, thermal treated sludge has conspicuous discrepancy in the solid concentration.

2.2.3 SLUDGE CAKE AND FILTRATE

When rated by moisture concentrations included in the dewatered cake, the filter press comes the first place. This can yield easily the cake of which moisture content is less than 50 percent. The running up is the centrifuge or the vacuum filter, depending on local conditions (Table 2.6).

Suspended solid concentration in filtrate produced by the vacuum filter is generally less than that by centrifuge. The concentrations is no greater than 1000 mg/l, except some extra ordinal cases. Seven cases that the SS concentration in the filtrate exceeds 3000 mg/l are observed.

2.2.4 CONDITIONING

Lime and ferric chloride are the most commonly used chemicals for the purpose of sludge conditioning in advance of either the vacuum filter or the filter press: 101 out of the 135 vacuum filters are fed with the sludge which is conditioned by using the lime and ferric chloride and so does 6 out of the 13 filter press.

On the basis of a dry solid, 25 percent and 10 percent are average feeding rates of the lime and the ferric chloride respectively. When lime dosage increases, proportionally the ferric chloride dosage increases (Table 2.7). Even for conditioning digested sludge, the same amount of these chemicals as for conditioning the undigested sludge is widely used.

Polymer is the only chemicals that are used to condition sludge for the centrifuge. There is no plant where inorganic coagulant is dosed for centrifuge purposes.

2.2.5 PERFORMANCE

Comparisons of design capacities and actual performances in 108 vacuum filter plants are shown on Table 2.8. The case that an amount of sludge actually handled is greater than the originally designed capacity is observed in 28 vacuum filter plants and 4 filter press plants.

One of the difficult decisions in selecting the centrifuge is how to satisfy both efficiency in SS recovery and efficiency in dewatering. Usually an increase in the SS recovery results in an decrease in the dewatering efficiency. If the fed sludge concentration is more than 20,000 mg/l, more than 80 percents SS recovery rate is expected.

61 dewatering plants are operated between 3 and 6 hours daily, 35 are so less than 3 hours and 36 are between 6 and 12 hours. Only 11 plants are operated for 24 hours every day.

An amount of raw sludge produced is closely related to an amount of raw sewage to be treated. A correlation coefficient between the amounts of raw sewage and raw sludge is 0.61 (Fig. 3.1). Every one cubic meter of the raw sewage treated, about 20 liters of the raw sludge are produced during the course of the primary and the secondary treatment.

The raw sewage rate to be treated and the power consumed by either of the three dewatering equipments are in a linear correlation (Fig. 3.2). A magnitude of its tangential value is different, depending on the equipments; 0.037 KWH/m^3 for the vacuum filter, 0.0123 KWH/m^3 for the centrifuge, and 0.0546 KWH/m^3 for the filter press. A energy consumption per unit solid is greatest for the filter press, middle for the vacuum filter, and least for the centrifuge.

Table 2-1 Relationship between Design Capacity and Process

Process / Design capacity (m^3)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	Total
0 ~ 5,000	14	0	1	0	2	0	3	0	1	0	0	0	0	0	21
5,000 ~ 10,000	11	0	1	0	4	0	4	0	0	0	0	0	0	1	21
10,000 ~ 20,000	6	0	4	0	3	1	4	0	0	0	0	0	0	0	18
20,000 ~ 50,000	8	2	5	1	19	1	7	0	0	2	1	1	1	0	48
50,000 ~ 100,000	10	0	1	1	10	1	1	4	0	0	0	0	0	0	28
100,000 ~ 200,000	6	0	0	0	5	0	0	3	0	0	0	1	0	0	15
200,000 ~ 500,000	8	2	0	0	4	0	0	2	0	0	0	0	0	0	16
500,000 ~	1	0	0	0	2	0	0	0	0	0	0	0	0	0	3
Total	64	4	12	2	49	3	19	9	1	2	1	2	1	1	170

Note:
1 : Conventional activated sludge process
2 : Modified aeration process
3 : High rate aeration sedimentation process
4 : 1 & 3
5 : Step aeration process
6 : 1 & 5
7 : Trickling filter process
8 : Plain sedimentation
9 : Single Stage Digestion
10 : Neutralization
11 : 3 & 7
12 : 5 & 8
13 : 1 & 7
14 : Others

Table 2-2 Relationship between Design Capacity and Type of Dewatering Machine

Type of dewatering machine Design capacity (m^3)	Vacuum filter	Centrifuge	Filter press	Others	Total
0 ~ 5,000	17	3	0	1	21
5,000 ~ 10,000	17	3	1	0	21
10,000 ~ 20,000	13	5	0	0	18
20,000 ~ 50,000	42	3	3	0	48
50,000 ~ 100,000	19	6	3	0	28
100,000 ~ 200,000	11	0	4	0	15
200,000 ~ 500,000	13	1	2	0	16
500,000 ~	3	0	0	0	3
Total	135	21	13	1	170

Table 2-3 Relationship between Treatment System and Capacity

Treatment system \ Capacity (average) m3/day	0 ~ 5,000	5,000 ~ 10,000	10,000 ~ 20,000	20,000 ~ 50,000	50,000 ~ 100,000	100,000 ~ 200,000	200,000 ~ 500,000	500,000 ~	Total
1-2-3-4	6	2	6	14	7	4	2	2	43
1-2-3	1	2	3	5	3	1	2	0	17
1-2-4	0	0	0	0	1	1	0	0	2
1-2	5	4	1	2	1	3	2	0	18
1-3-4	0	0	0	0	0	0	0	0	0
1-3	0	0	0	0	0	0	0	0	0
1-4	3	1	0	1	1	1	1	0	8
1	15	2	8	14	10	2	0	0	51
2-3-4	2	2	1	0	1	0	0	0	6
2-3	1	0	1	2	3	1	0	0	8
2-4	0	0	0	0	0	0	0	0	0
2	1	2	2	1	1	0	0	0	7
3-4	0	0	0	0	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0
Others	7	0	1	0	2	0	0	0	10
Total	41	15	23	39	30	13	7	2	170

Note: 1 : Thickner
2 : Digestion tank
3 : Sludge elutriation tank
4 : Sludge storage tank

Table 2-4 Relationship between Treatment System and Type of Dewatering Machine

Type of dewatering machine Treatment system	Vacuum filter	Centrifuge	Filter press	Others	Total
1-2-3-4	39	2	2	0	43
1-2-3	16	0	1	0	17
1-2-4	2	0	0	0	2
1-2	13	5	0	0	18
1-3-4	0	0	0	0	0
1-3	0	0	0	0	0
1-4	6	1	1	0	8
1	38	4	8	1	51
2-3-4	6	0	0	0	6
2-3	1	1	0	0	8
2-4	0	0	0	0	0
2	3	4	0	0	7
3-4	0	0	0	0	0
3	0	0	0	0	0
4	0	0	0	0	0
Others	5	4	1	0	10
Total	135	21	13	1	170

Note:
1 : Thickner
2 : Digestion tank
3 : Sludge elutriation tank
4 : Sludge storage tank

Table 2-5 Comparison of Solid Concentration between Digested and Raw Sludge

Solid concentration (%)	Digestion	No-digestion	Total
0 ~ 1	0	0	0
1 ~ 2	1	0	1
2 ~ 3	10	9	19
3 ~ 4	17	10	27
4 ~ 5	21	13	34
5 ~ 6	17	14	31
6 ~ 7	8	3	11
7 ~ 8	6	7	13
8 ~ 9	3	0	3
9 ~ 10	1	0	1
10 ~ 15	5	2	7
15 ~ 20	0	1	1
20 ~ 25	0	1	1
25 ~	0	0	0
No data	9	12	21
Total	98	72	170

Table 2-6 Cake Solid Concentration after Dewatering

Type of dewatering machine Cake solid concentration (%)	Vacuum filter	Centrifuge	Filter press	Others	Total
0 ~ 15	0	0	0	0	0
15 ~ 20	10	3	1	1	15
20 ~ 25	44	6	1	0	51
25 ~ 30	39	6	0	0	45
30 ~ 35	14	2	1	0	17
35 ~ 40	8	0	3	0	11
40 ~ 45	0	0	0	0	0
45 ~ 50	1	0	3	0	4
50 ~ 55	0	0	1	0	1
55 ~	1	0	2	0	3
No data	18	4	1	0	23
Total	135	21	13	1	170

Table 2-7 Dose of Fe^{3+} and Ca^{2+} in Vacuum Filter

Fe^{3+} dose (%) \ Ca^{2+} dose (%)	0 ~ 10	10 ~ 20	20 ~ 30	30 ~ 40	40 ~ 50	50 ~ 60	60 ~ 70	70 ~ 80	80 ~ 90	90 ~ 100	100 ~	Total
0 ~ 5	19	10	6	2	2	2			1			42
5 ~ 10	1	13	11	10	1	2						38
10 ~ 15		1	5	5	5	2						18
15 ~ 20				2				1				3
20 ~ 25				1		2					1	4
25 ~ 30				2		1						3
30 ~ 40									1			1
40 ~ 50						1						1
50 ~											1	1
Total	20	24	22	22	8	10	0	1	2	0	2	111

No data 24

Table 2-8 Relationship between Solid Concentration and Yield of Vacuum Filter

Solid Conc.(%) \ Actual Yield / Design Yield (%)	0 ~ 10	10 ~ 20	20 ~ 30	30 ~ 40	40 ~ 50	50 ~ 60	60 ~ 70	70 ~ 80	80 ~ 90	90 ~ 100	100 ~	Total
0 ~ 1												0
1 ~ 2											1	1
2 ~ 3	1		1	1	5		1	1			1	11
3 ~ 4		1	2	3	2	4	3	1		1	3	20
4 ~ 5		1	4		2	4	5	2	1	1	4	24
5 ~ 6	1			2	5		2	3	1	1	8	23
6 ~ 7	1			1	1			2	1		2	8
7 ~ 8		2	1			1			2	1	4	11
8 ~ 9											2	2
9 ~ 10											1	1
10 ~				1	1		1		2		2	7
Total	3	4	8	8	16	9	12	9	7	4	28	108

No data 27

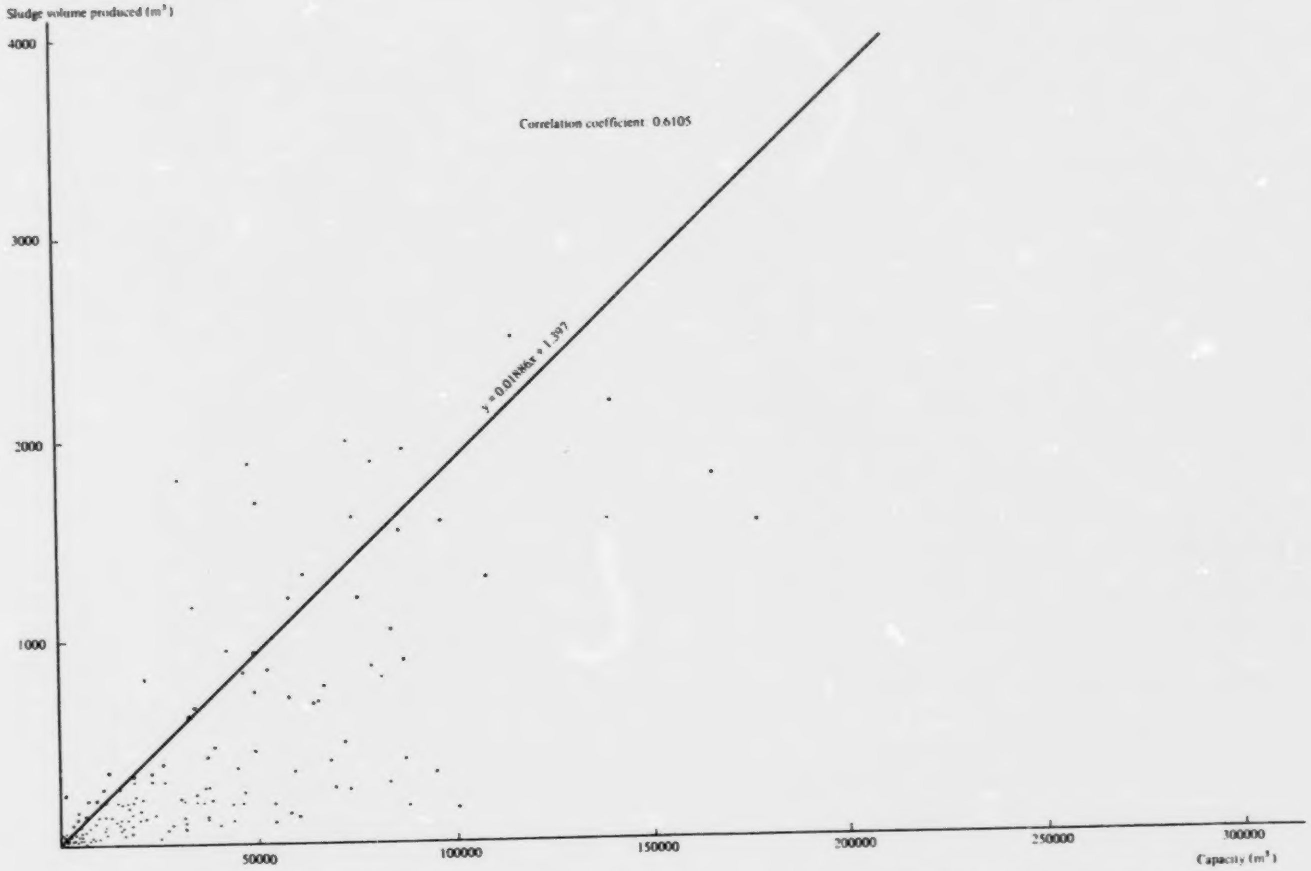


Fig. 2-1 Relationship Between Sludge Volume and Capacity (Vacuum filter)

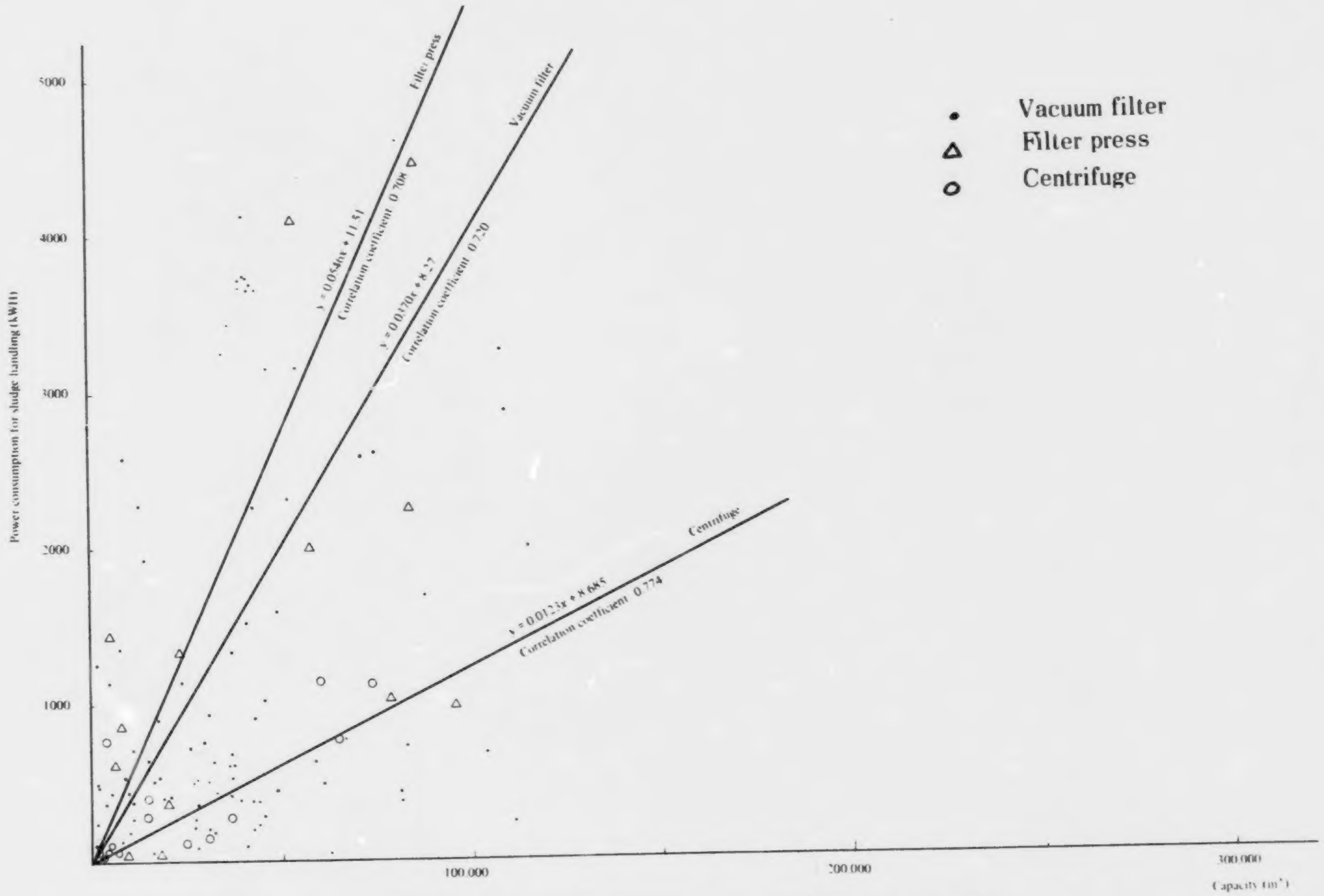


Fig. 2-2 Relationship Between Capacity and Power Consumption

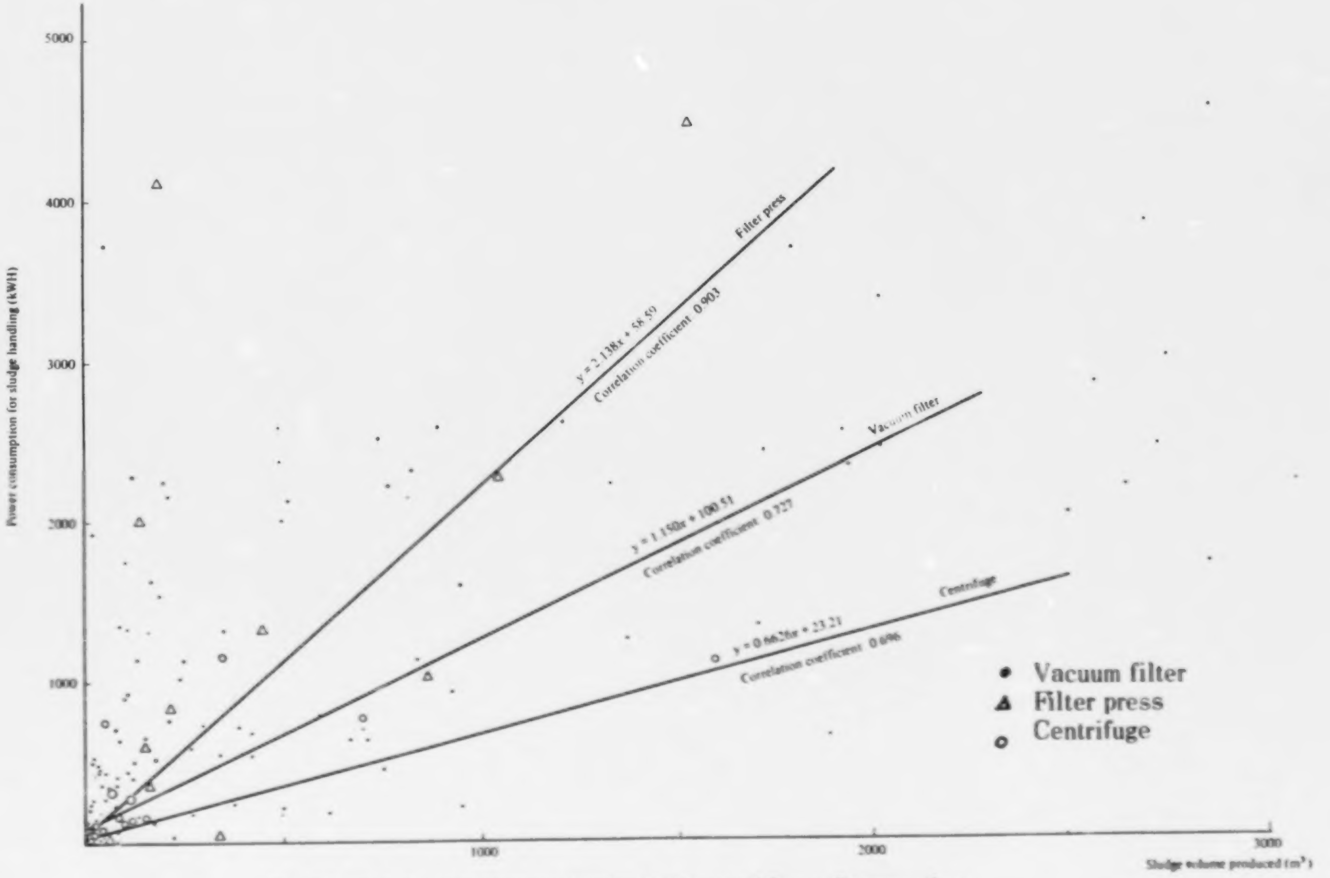


Fig. 2-3 Relationship Between Sludge Volume and Power Consumption

CHAPTER 3

SEWAGE SLUDGE TREATMENT AND DISPOSAL AS PRACTICED IN YOKOHAMA CITY

– "RECLAMATION TO AGRICULTURAL LAND AND GREEN FIELD" PROGRAM –

CONTENTS

1. Present Status of Sewage Sludge Treatment/Disposal 26
2. Problems Associated with Sewage Sludge Disposal 28
3. "Reclamation to Agricultural Land and Green Field" Program 28
   3.1 Size of "Reclamation to Agricultural Land and Green Field" Program .. 29
   3.2 Sludge Dryer Installation 29
   3.3 Outline of Installation 31
   3.4 Construction and Operation/Maintenance Costs 31
   3.5 Properties of Dried Sludge 32

Sewage Sludge Treatment and Disposal as Practiced in Yokohama City

– "Reclamation to Agricultural Land and Green Field" Program –

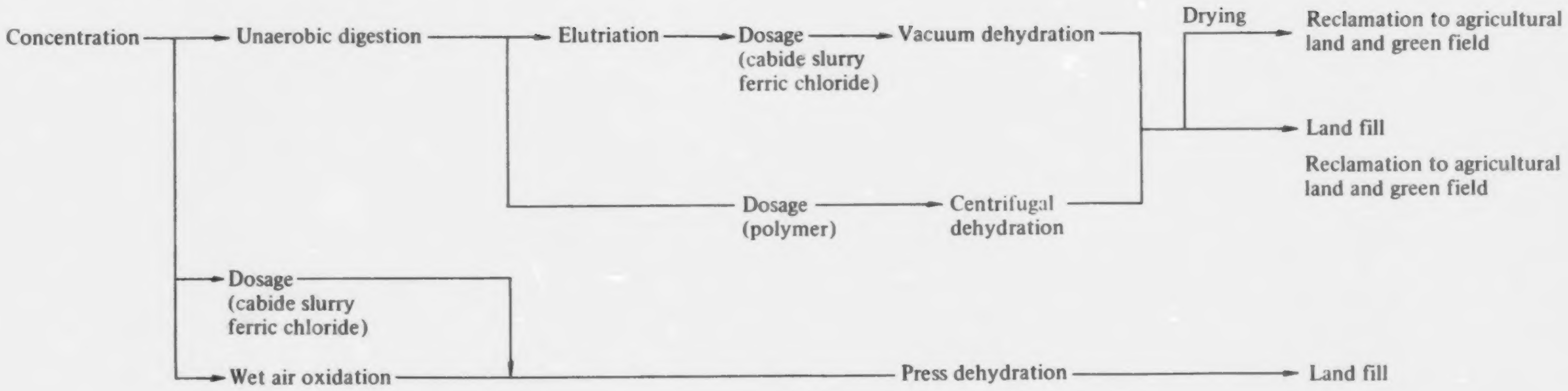
1. PRESENT STATUS OF SEWAGE SLUDGE TREATMENT/DISPOSAL

In the sewerage plan of Yokohama City, constructing ten sewage treatment plants in nine treatment districts is planned, and so far five are already in service. At these treatment facilities, secondary treatment by activated sludge process is performed, and the sludge produced from these facilities today amount to, in terms of concentrated sludge (moisture content 96%), roughly 600,000 cubic meters annually. Treatment and disposal methods practised are: (1) unaerobic digestion process at two plants and wet air oxidation at one plant, followed by (2) dehydration in both cases, and (3) disposed to, in most part, dumping yards of the City as land fill.

Characteristics of sludge cakes: those treated by unaerobic digestion process has a moisture content of 70 to 75% (carbide slurry and ferric chloride added) with volatile matter 25 to 30%; those by wet air oxidation has a moisture content of about 40% with volatile matter 10 to 15%.

Given in Fig. 3–1 is the outline of sewage sludge treatment/disposal process and major sludge treatment facilities.

Fig. 3–1 Sewage sludge treatment and disposal process


Concentration
Unaerobic digestion
Elutriation
Dosage
(cabide slurry
ferric chloride)
Vacuum dehydration
Drying
Reclamation to agricultural land and green field
Land fill
Reclamation to agricultural land and green field
Dosage
(polymer)
Centrifugal dehydration
Dosage
(cabide slurry
ferric chloride)
Wet air oxidation
Press dehydration
Land fill

2. PROBLEMS ASSOCIATED WITH SEWAGE SLUDGE DISPOSAL

The volume of sewage sludge produced in Yokohama City roughly doubled in three years since 1970. Sludge cakes produced presently amounts to 40,000 cubic meters annually, and it is estimated to increase by tenfold, or 400,000 m^3, by 1985.

This sludge is now, as it was described earlier, dumped over dumping yards within the City, however, due to rapid expansion of dwelling districts, distant dumping yards are being sought, and with increased traffic conditions within the City, its transportation is found more and more difficult these days. Moreover, it is very difficult to find new inland dumping yards today. And at the present dumping yards, disposed cakes tend to become a very soft mud as they quickly absorb a large quantity of water, posing a troublesome problem to proper land fill operations. Difficulties in handling, bad-smelling and leachate out of the sludge cake are among the associated problems for which we are under the pressure to find other means of disposal.

3. "RECLAMATION TO AGRICULTURAL LAND AND GREEN FIELD" PROGRAM

It is the principal rule to follow, in its end, that all sewage sludge is returned to the ground by accomplishing it in harmony with the Nature's cyclic process of materials. It is desirable, therefore, to dispose sewage sludge to the ground or to the ocean, in a manner that will not disturb, or helps the ecosystem of the Nature.

For a coastal city like Yokohama, although we cannot choose but to rely on returning it to the ocean and reclaiming a foreshore while concentrating our efforts on finding inland dumping yards, we must work to establish a disposal scheme primarily based on returning to the ground and reclamation to agricultural and green field, as a more essential solution to this problem.

Since sewage sludge contains nitrogen and phosphorus and is rich with organic matter, unlike chemical fertilizers, when it is returned to the soil, its benefit is not limited to serving as an enrichener, but to aid in producing an environment necessary for the growth of bacteria in the soil, subsequently creating an airy soil construction desirable for vegitation with good retention of moisture.

A three year study, from 1971 to 1974, that Yokohama City conducted in cooperation with the Tokyo University of Education reveals that it could be a hopeful fertilizer with about the same dressing effectiveness as matured compost, whereas it is slow acting to dry field rice plant and barley when compared with chemical fertilizers.

As for its dressing effectiveness to cyclamen, a garden plant, it is found to be much the same as that of the soil customarily used, it is a slow-acting but long-standing fertilizer. When sludge is used to lawn soil, it is found that lawn's growth in height and in the area of its leaves increased with the increase of its sludge appliction.

For acceptable application of sewage sludge to the soil, ease of handling, storage and transportation, and freedom from discomfort are required. Pathogenic bacteria and weed seeds must be sterilized, too.

For these reasons, it is desirable that, following dehydration processing, sludge be dried by dryers to reduce its moisture content further until granular sludge is obtained.

In Yokohama City, operational sludge dryers were installed in the Nambu Sewage Treatment Plant in 1973 and 1974 as part of promoting "the reclamation to agricultural land and green field" program.

3.1 SIZE OF "RECLAMATION TO AGRICULTURAL LAND AND GREEN FIELD" PROGRAM

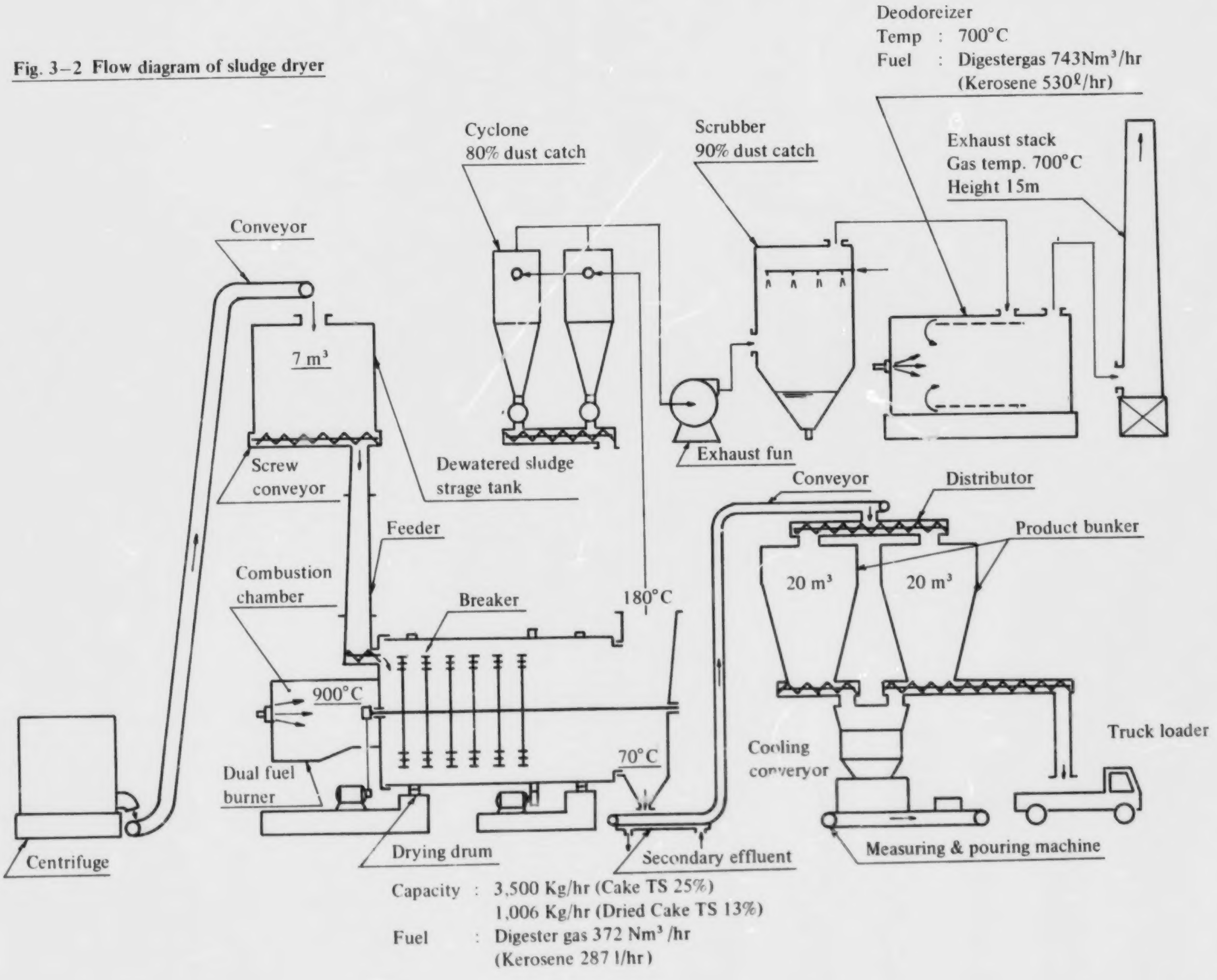
Before sludge reclamation to agricultural land and green field is carried out, such factors as secondary public nuisance by heavy metals, or plants' absorption of heavy metals and their accumulation in the soil, must be thoroughly studied. Sludge application therefore should be carefully selected, and for the time being, limiting its application to greenyard of parks in the City, an annual desposition of 2,600 tons of dry sludge (equivalent to roughly 24% of the sludge cakes produced) is planned.

3.2 SLUDGE DRYER INSTALLATION

Digested sludge is added with polymer and then dehydrated by a centrifuge before it is fed into the sludge dryer system.

The project size of installation: two sludge dryer systems to be run on a six-hour-a-day basis, each with a capacity of 1,300 tons of dried cludge output. One system is to be installed initially. If the installation is operated on a full scale, it would have the capability of producing some 10,000 tons of dried sludge annually. The drying system is schematically represented in Fig. 3–2.

Fig. 3-2 Flow diagram of sludge dryer



Conveyor
7 m³
Screw conveyor
Dewatered sludge strage tank
Feeder
Combustion chamber
900°C
Dual fuel burner
Centrifuge
Breaker
Drying drum
180°C
70°C
Cyclone
80% dust catch
Scrubber
90% dust catch
Exhaust fun
Exhaust stack
Gas temp. 700°C
Height 15m
Conveyor
Distributor
Product bunker
20 m³
20 m³
Cooling converyor
Secondary effluent
Measuring & pouring machine
Truck loader

Deodoreizer
Temp : 700°C
Fuel : Digestergas 743Nm³/hr
(Kerosene 530ℓ/hr)

Capacity : 3,500 Kg/hr (Cake TS 25%)
1,006 Kg/hr (Dried Cake TS 13%)
Fuel : Digester gas 372 Nm³ /hr
(Kerosene 287 l/hr)

3.3 OUTLINE OF INSTALLATION

The drying system consists essentially of the following four elements:

a) Sludge Feeder

Dehydrated sludge is continuously fed to the dryer with a variable-speed screw conveyor in a batch process.

(b) Dryer

While moving slowly in the axial direction, sludge is exposed to heated air of about 900°C blown in the same direction as the drum rotates slowly. Temperatures within the drum are so controlled that it is approximately 900°C at the entrance; about 120°C at the outlet.

Sludge is scraped upward to the upper part of the drum with blades attached on the inner wall of the drum, and is crushed by rotating rods as it falls off from the upper part of the drum. By this repeated process of crushing, sludge gets greater surface area while moving from one end to the other in the drum, and it contacts with the hot air for about 40 minutes. Obtained is granular sludge with a moisture content of about 13%.

For the best possible utilization of energy, primarily digestion gas is used as fuel-another benefit of eliminating air pollution by other fuels.

(c) Dry Sludge Storage/Bagging Unit

An automatic measuring/bagging machine packages dry sludge in polyethylene bags in a 20 kg batch.

(d) Exhaust Gas Clarifyer

Cyclone dust catchers and a scrubber are arranged in series. With 98% of dust caught by this arrangement, the dust content of air released to the atmosphere is 0.17 g/Nm^3, or 60% below the 0.4 g/Nm^3 requirement specified under the Kanagawa Prefecture Public Nuisance Prevention Act.

Deodorization of exhaust gas is accomplished through oxidation-decomposition by heating it to about 700°C. The deodorization furnace temperature is controlled to 700°C. At this temperature, nitrogen oxides generation is reduced to a minimum.

Exhaust gas is free of sulfur oxides when digestion gas is used. Their discharge with the use of kerosene is 2.245 m^3/h, 50% below the 4.1 Nm^3/h requirement.

Calories necessary for the deodorization are: with digestion gas, 4,454,000 kCal per hour with a fuel consumption of 743 Nm^3 per hour; with kerosene, 530 liters per hour.

3.4 CONSTRUCTION AND OPERATION/MAINTENANCE COSTS

Listed below is the breakdown of construction cost of the sludge dryer installation:

Dryer system	165,900,000 yen
Electrical system	80,400,000 yen
Buildings	108,384,000 yen
Miscellaneous	1,390,000 yen
Total	356,074,000 yen

Annual cost for operating the dryer installation on a 3-personnel operator, 6-daytime-hour run basis is estimated as follows:

Personnel cost	9,000,000 yen
Electricity cost	2,202,000 yen
Repairs, expendables	6,900,000 yen
Equipment lease (fork trucks)	1,200,000 yen
Depreciation of installation	15,027,000 yen

Fuel cost	0 yen
Total	34,329,000 yen

With an annual dried sludge production of 1,300 tons, the dehydrated sludge costs about 7,600 yen per ton.

3.5 PROPERTIES OF DRIED SLUDGE

Composition of sludge before and after drying processing as a fertilizer, is given in Tables 3–1. You may notice that components remain much the same before and after processing in the dryer. Dry sludge generally has a moisture content of 5 to 16% with granular sizes ranging between 1.0 and 3.3 millimeters in diameter.

Tables 3–1 Compositon of sludge before and after drying processing as a fertilizer

Item	Dehydrated Sludge	Dried Sludge	Note
Total Solids	23.8%	90.0%	Weight % of Sludge
Volatile Solids	39.7	35.8	Weight % of Solid
Phosphorus (P_2O_5)	3.2	3.1	Weight % of Solid
Patassium (K_2O)	0.77	0.75	Weight % of Solid
Calcium (CaO)	2.9	3.0	Weight % of Solid
Magnesium (MgO)	1.0	1.0	Weight % of Solid
Nitrogen (Kjeldahl)	3.3	2.9	Weight % of Solid
pH	7.45	7.17	Weight % of Solid

CHAPTER 4. THE TREATMENT AND DISPOSAL OF SLUDGE AT TOBA TREATMENT PLANT

4.1 Introduction 34
4.2 Outline of Sludge Treatment Facilities 34
4.3 Operation Data of Sludge Treatment Facilities 36
  Solids Balance at Each Treatment Process 36
  Thickening Tanks 38
  Digestion Tanks 39
  Dewatering Facilities 40
  Incinerators 41
4.4 Some Considerations on Toxic Substances and Heavy Metals in Disposing Sludge 42
  Toxic Substances in the Exhaust Gas from the Incinerators and the Countermeasures 42
  Effluence Test of Toxic Substances and Others from Ash 42
4.5 Researches on Sludge Ash for its Reuse 45
  Utilization for Soil Conditioners 45
  Use as a Material for Road Construction 45

4.1 INTRODUCTION

Kyoto is an inland city located on the midstream of the Yodo River. On the lower reaches of the river there is a group of cities centering around Osaka, and the river is the only source of water for approximately ten million people in those cities. The river water, therefore, is widely used for drinking, farming, and industries.

Aiming to ameliorate the life environment of the citizens and to prevent water pollution of the Yodo river, the City of Kyoto has been endeavoring to construct its sewerage system. As of March 1975, the city's sewer system covers 45% of the urbanized area of the city. That is, 48% of the total citizens share in the benefit of the system. By the year 1985 the system is to cover the whole of the urbanized area of the city.

At present the sewage is treated by three plants, namely Toba, Kisshoin, and Fushimi Plant. (Table 4.1) Of the three, the Toba Plant is the biggest. The present conditions of the sludge treatment at Toba Plant are described below.

Table 4.1 Present Conditions of the Three Plants

Name of Plant	Treat't Capacity m^3/d	Qty. of Inflow m^3/d	Treat't Process	Sludge Treatment Process
Toba	570,000	388,400	Activated Sludge Process	Thickening-Anaerobic Digestion-Vacuum Filtration - Incineration-Reclamation or Thickening-Vacuum Filtration
Kisshoin	92,900	94,300	Activated Sludge Process	Pumping to Toba Plant
Fushimi	55,000	22,100	Activated Sludge Process	Thickening-Vacuum Filtration-- Transport to Toba Plant

Note: The quantities listed above are the average inflow at the time of dry weather during the period from April 1974 to March 1975

4.2 OUTLINE OF SLUDGE TREATMENT FACILITIES

At the Toba Plant, not only the sludge produced at the Plant but the night soil brought into the digestion tanks and the sludge pumped from the Kisshoin Plant are treated. As shown by Figure 4.1, before the sludge becomes ash and is used for landfill, it is digested, dewatered and incinerated, or raw sludge is dewatered and incinerated. The sludge cake dewatered at the Fushimi Plant is also transported to the Toba Plant, where it is incinerated. Thus all the sludge produced at those three plants is in the ultimate treated and disposed of at the Toba Plant.

Table 4.2 shows an outline of the sludge treatment facilities.

Figure 4.1 Flow Diagram of Sludge Treatment

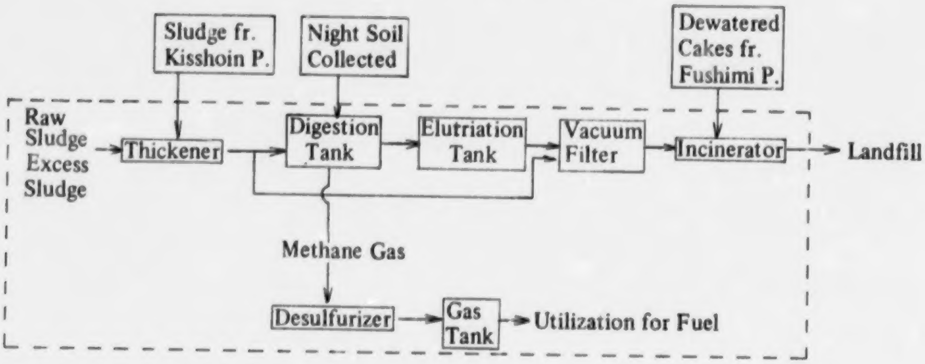

Sludge fr. Kisshoin P.
Night Soil Collected
Dewatered Cakes fr. Fushimi P.
Raw Sludge
Excess Sludge
Thickener
Digestion Tank
Elutriation Tank
Vacuum Filter
Incinerator
Landfill
Methane Gas
Desulfurizer
Gas Tank
Utilization for Fuel

Table 4.2 Outline of the Sludge Treatment Facilities

1. Sludge Thickening Tanks

	I	II	III	IV
Type	Circular Tank	Square Tank	Circular Tank	
Diameter	20.0m	17.0m x 17.0m	20.0m	
Water Depth at Wall	3.0m	3.6m	3.0m	
Number of Tanks	2	2	4	
Capacity, Each Tank	966m³	1,160m³	942m³	

2. Sludge Digestion Tanks (Two Stage Digestion)

	I	II	III	IV
Type	Cylindrical Tank with Cone Bottom and Fixed Cover			
Diameter	25.0m	25.0m		25.0m
Water Depth at Wall	5.2m	8.2m		8.2m
Water Depth at Center	9.2m	11.1m		11.1m
Heating System	Heat Exchanger			Steam Blowing
Agitating System	Gas Agitator			
Number of Primary Digestion Tanks	1	2	3	3
Number of Secondary Digestion Tanks	1	2	1	1
Capacity, Each Tank	3,600m³ 3,090m³	4,400m³		
Temperature	30°C	30°C		
Detention Period	30 days	30 days		22.5 days

3. Vacuum Filters

	I	II
Type	Belt Filter	
Diameter of Drum	3.5m	
Length of Drum	4.2m	
Surface Area of Filter	47m^2	
Rate of Filtration	17.5 kg dry solid/m^2·hr	
Number Installed	8	12

4. Incinerators

Type	Multiple-Hearth Type		
Outside Diameter	4.35m	5.10m	6.78m
Height	10.04m	11.34m	12.35m
Burning Temperature	800°C		
Number of Stages	8		
Capacity, Each	60t sludge cake/day		150t sludge cake/day
Number Installed	2	1	3

4.3 OPERATION DATA OF SLUDGE TREATMENT FACILITIES

4.3.1 SOLIDS BALANCE AT EACH TREATMENT PROCESS

The average daily figures of solids balance at each treatment process at the Toba Plant during the period from April 1974 to March 1975 are shown by Figures 4.2, 4.3, and 4.4. The suspended solids in the inflowing sewage is 57 tons. Through the sewage treatment process, 14.7 tons of excess activated sludge is synthesized from the soluble matter. Further the solids in the wastewater to be returned to the sewage treatment facilities from the sludge treatment facilities are added to it. Thus the total quantity of the sludge solids that are sent to the thickening tanks becomes 127.3 tons.

Besides the aforementioned, 9 tons of sludge solids is sent to the thickening tanks from the Kisshoin Plant, 6.5 tons of solids in the night soil is thrown into the digestion tanks, and 2.3 tons of dewatered cakes is sent to the incinerators from the Fushimi Plant. Thus all the sludge produced at those three plants is treated and disposed of by the Toba Plant.

As shown by Figure 4.4, altogether 136.3 tons of solids is sent to the thickening tanks, and out of the 126 tons efflux 57.2 tons is fed to the digestion tanks and the other 68.8 tons is sent to the dewatering facilities. Of the sludge solids fed to the digestion tanks, 12.5 tons is gasified and other 30.5 tons is sent to the dewatering facilities as digested sludge. By the incinerators, altogether 360.4 tons of dewatered cakes is incinerated and 48.1 tons of ash is produced. The 360.4 tons of cakes consist of 261.6 tons of raw sludge cakes, 88.8 tons of digested sludge cakes, and 10 tons of cakes sent from the Fushimi Plant.

The solids in the wastewater from the sludge treatment facilities are 10.3 tons from the thickening tanks, 20.7 tons from the digestion tanks, and 29.6 tons from the dewatering facilities.

Figure 4.2 BOD and SS Balance at the Sewage Treatment Process

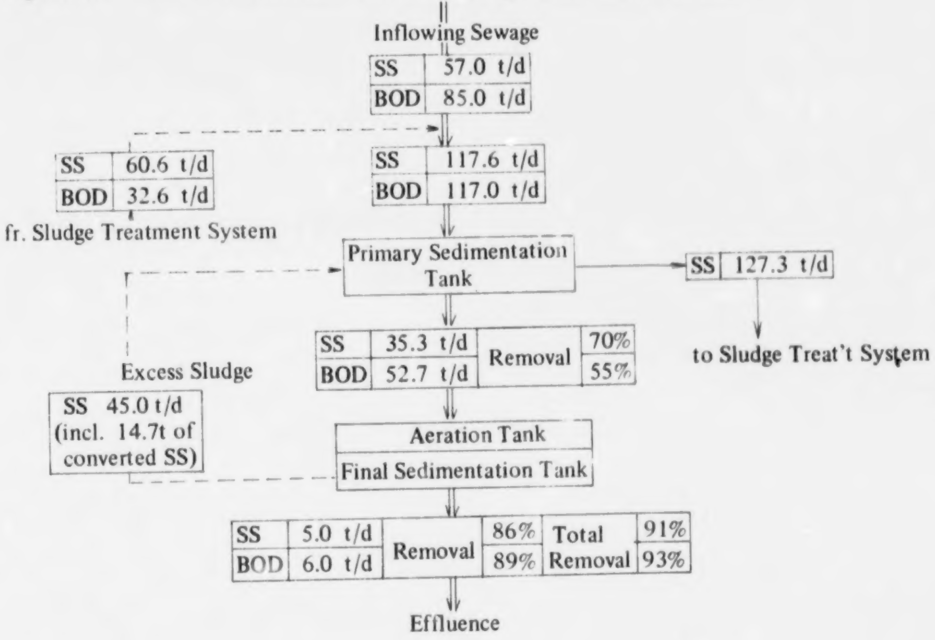

Inflowing Sewage
SS 57.0 t/d
BOD 85.0 t/d
SS 60.6 t/d
BOD 32.6 t/d
fr. Sludge Treatment System
SS 117.6 t/d
BOD 117.0 t/d
Primary Sedimentation Tank
SS 127.3 t/d
to Sludge Treat't System
Excess Sludge
SS 45.0 t/d (incl. 14.7t of converted SS)
SS 35.3 t/d
BOD 52.7 t/d
Removal
70%
55%
Aeration Tank
Final Sedimentation Tank
SS 5.0 t/d
BOD 6.0 t/d
Removal
86%
89%
Total Removal
91%
93%
Effluence

Figure 4.3 Solids Balance at Toba Plant

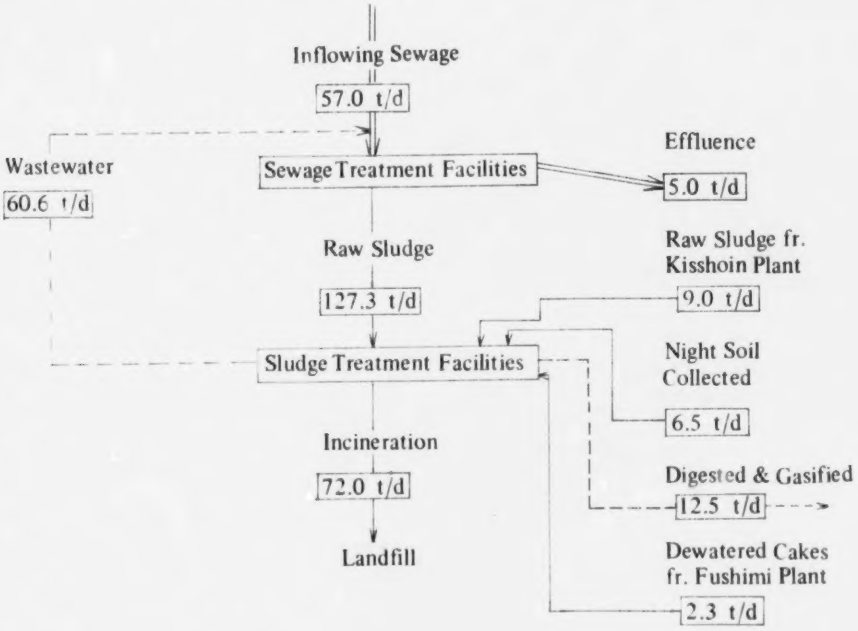
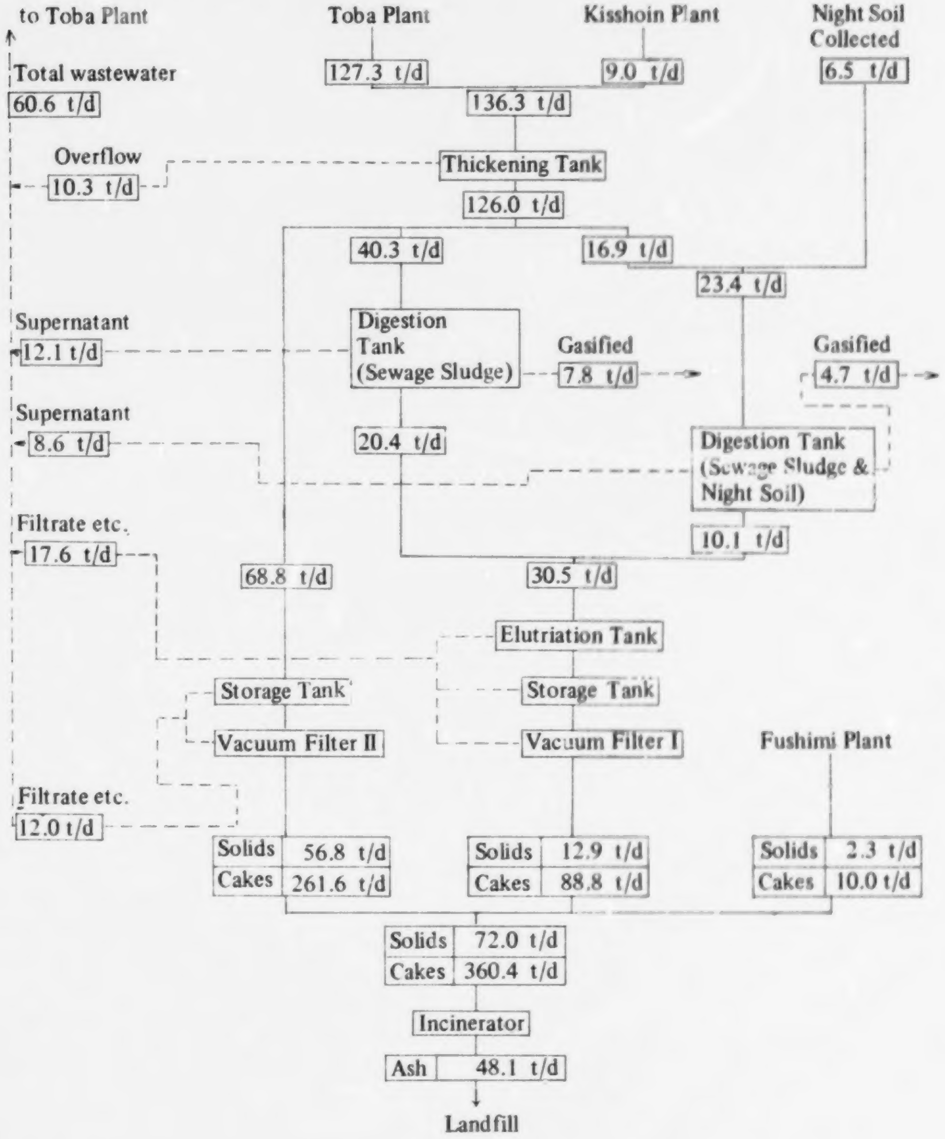

Inflowing Sewage
57.0 t/d
Wastewater
60.6 t/d
Sewage Treatment Facilities
Effluence
5.0 t/d
Raw Sludge
127.3 t/d
Raw Sludge fr. Kisshoin Plant
9.0 t/d
Sludge Treatment Facilities
Night Soil Collected
6.5 t/d
Incineration
72.0 t/d
Digested & Gasified
12.5 t/d
Landfill
Dewatered Cakes fr. Fushimi Plant
2.3 t/d

Figure 4.4 Solids Balance at the Sludge Treatment Process


to Toba Plant
Total wastewater
60.6 t/d
Toba Plant
127.3 t/d
Kisshoin Plant
9.0 t/d
Night Soil Collected
6.5 t/d
136.3 t/d
Overflow
10.3 t/d
Thickening Tank
126.0 t/d
40.3 t/d
16.9 t/d
23.4 t/d
Supernatant
12.1 t/d
Digestion Tank (Sewage Sludge)
Gasified
7.8 t/d
Gasified
4.7 t/d
Supernatant
8.6 t/d
20.4 t/d
Digestion Tank (Sewage Sludge & Night Soil)
10.1 t/d
Filtrate etc.
17.6 t/d
68.8 t/d
30.5 t/d
Elutriation Tank
Storage Tank
Storage Tank
Vacuum Filter II
Vacuum Filter I
Fushimi Plant
Filtrate etc.
12.0 t/d
Solids 56.8 t/d
Cakes 261.6 t/d
Solids 12.9 t/d
Cakes 88.8 t/d
Solids 2.3 t/d
Cakes 10.0 t/d
Solids 72.0 t/d
Cakes 360.4 t/d
Incinerator
Ash 48.1 t/d
Landfill

4.3.2 THICKENING TANKS

The thickening tanks are divided into four groups. The operation data of the tanks are shown by Table 4.3. The average concentration of the solids in the sludge coming from the primary sedimentation tanks is 1.1 to 1.5%. The average concentration of the solids in the thickened sludge is 2.1 to 2.6%. Figure 4.5 shows the relationship between the initial sludge concentration and the rate of the concentration increase. It shows how poor the thickening efficiency is. At the Toba Plant the efficiency is gradually lowering in recent years.

Figure 4.5 Relationship between the Initial Sludge Concentration and the Rate of the Concentration Increase

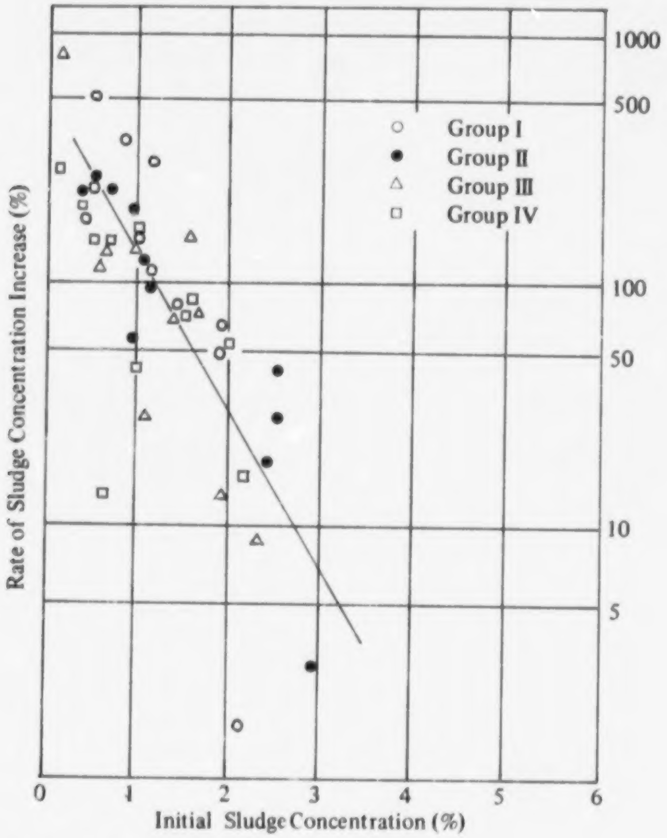


Table 4.3 Operation Data of Sludge Thickeners

Gr.	Raw Sludge		Thick Sludge		Effluent			Solids Loading kg/m²·d	Overflow Rate m³/m²·d	Detent. Period hrs.
	Solids t/d	Conc. %	Solids t/d	Conc. %	Solids t/d	TS mg/l	BOD mg/l			
I	35.6	1.1	34.0	2.6	1.6	474	185	54.2	8.5	8.9
II	39.7	1.5	36.7	2.5	3.0	506	152	68.7	13.3	7.3
III	35.4	1.4	30.9	2.1	4.5	1,530	579	56.8	8.2	9.8
IV	25.6	1.4	24.4	2.1	1.2	610	176	40.3	5.3	15.0
Total	136.3		126.0		10.3					

4.3.3 DIGESTION TANKS

The digestion tanks are divided into four groups. Every one of those tanks is equipped with a heating system and a gas agitator. The operation data of the digestion tanks are shown by Table 4.4. It must be noted that the digestion conditions of Group III tanks are different from the others, because in these Group III tanks night soil is mixed with sewage sludge before digestion.

At Group IV tanks the heating is done in the tanks with steam blowing, while the tanks of the other Groups employ the heating by the duplex pipe counter flow heat exchangers. The temperature in the tank is kept at 27 to 30°C. The gas agitator is intermittently operated when sludge is fed in.

The digestion period is 23 to 30 days. The load of volatile solids is controlled 1.1 to 1.6 kg/m³·d.

It is previously mentioned that the concentration of the sludge fed in is low. The same of the digested sludge is also low being only 3.0 to 3.2%. The volatile acids in the digested sludge at Groups I, II, and IV are 150 to 200 mg/l and at Group III it is 400 mg/l. The alkalinity in Groups I, II, and IV is 1,600 to 1,700 mg/l, while at Group III it is 3,800 mg/l. The BOD of the supernatant is 1,200 to 2,300 mg/l. In any of those tanks scum can hardly be found.

The digestion rate is low at every tank being approximately 30%. At these tanks, altogether approximately 12.5 tons of volatile solids is decomposed, and approximately 12,000 m³ of sludge gas is produced per day. After desulfurized, approximately 4,500 m³ of the sludge gas is utilized for fuel for the digestion tank heating, and other 7,500 m³ is used as supplementary fuel of sludge incinerators.

Table 4.4 Operation Data of Sludge Digestion Tanks

Gr.	Feeding Sludge			Digested Sludge					
	Solids t/d	Conc. %	Volatile Solids %	Solids t/d	Conc. %	Volatile Solids %	pH	Alkali. mg/l	Volatile Acids mg/l
I	7.6	2.6	65.2	4.5	3.0	58.3	7.2	1,649	196
II	15.4			9.5	3.2	57.4	7.2	1,706	149
III	16.9			10.1	3.0	59.3	7.6	3,828	396
	6.5	2.7	70.5						
IV	17.3	2.5	65.9	6.4	3.0	57.4	7.1	1,604	165
Total	63.7			30.5					

Gr.	Supernatant				Digest. Temper. °C	Digest. Period. days	Volatile Solids Loading kg/m³·d	Digest. Rate %	Gasif. Solids t/d
	Solids t/d	pH	TS mg/l	BOD mg/l					
I	1.7	7.3	6,338	1,519	26.8	27	1.5	29	1.4
II	3.0	7.3	5,347	1,159	27.7	37	1.2	29	2.9
III	8.6	7.7	11,249	2,311	28.8	23	1.1	29	4.7
IV	7.4	7.3	10,588	2,163	30.0	30	1.6	31	3.5
Total	20.7								12.5

4.3.4 DEWATERING FACILITIES

For sludge dewatering, the vacuum filters of the belt type have been employed. Eight filters of Group I dewater digested sludge and twelve filters of

Group II dewater raw sludge. The operation data of those dewatering facilities are shown by Table 4.5.

The concentration of the digested sludge to be dewatered is approximately 3%, while the same of raw sludge is approximately 2%. The alkalinity of the digested sludge is 1,600 to 3,800 mg/l, while the same of the elutriated sludge is approximately 600 mg/l. The elutriation of the digested sludge is done by two stage counter flow elutriation tank. The alkalinity of raw sludge is 460 mg/l.

As coagulants, ferric chloride ($FeCl_3$) and slaked lime ($Ca(OH)_2$) have been used. The chemical dosages to the sludge are 9.8% of $FeCl_3$ and 46.5% of $Ca(OH)_2$ for Group I, and 4.9% of $FeCl_3$ and 23.2% of $Ca(OH)_2$ for Group II.

The filter yield rate is 5.3 kg/m² .hr at Group I and 12.3 kg/m² .hr at Group II. The moisture content of the dewatered cakes is 77.1% at Group I and 72.5% at Group II. At Group I the filter yield rate is low and the moisture content is high. It is because the proper vacuum can not be retained owing to the following two reasons. Firstly the concentration of sludge is low and hair cracks are caused in the cakes on the filter medium at the time of dewatering due to the smallness of the quantity of fibrous materials. Secondly the efficiency of the facilities has been degraded due to superannuation.

The filter yield rate and the moisture content of the cakes are closely related with the sludge concentration. The low concentration of the sludge has caused the high dosage of the coagulants and the poor dewatering efficiency.

Table 4.5 Operation Data of Sludge Dewatering Facilities

Gr.	Feeding Sludge			Elutr. Sludge		Dewatered Cakes			$FeCl_3$ %	$Ca(OH)_2$ %	Filt. Yield Rate kg/m²·hr
	Solids t/d	Conc. %	Alkali. mg/l	pH	Alkali. mg/l	Qty t/d	Solids. t/d	Moist Cont.%			
I	30.5	3.1	2,392	6.9	607	88.8	12.9	77.1	9.8	46.5	5.3
II	68.8	2.2	----	6.5	461	261.6	56.8	72.5	4.9	23.2	12.3
Total	99.3					350.4	69.7				

4.3.5 INCINERATORS

All the dewatered cakes are incinerated. The incinerators are the multiple-hearth type. Three of the six incinerators incinerate 60 tons per day each, while the other three do 150 tons each.

The calorific value of dewatered cakes is 1,900 to 2,500 kcal/kg. As the supplementary fuel, heavy oil and sludge gas are utilized. For prevention of air pollution, low sulfureous heavy oil (sulfur content less than 0.8% and in winter the same less than 0.5%) is used, and 35 to 40 liters of the oil is consumed per ton of dewatered cakes. In case of digestion gas, whose calorific value is 5,600 kcal/m³, 50 to 70m³ of gas is consumed per ton of dewatered cakes.

The moisture content of the dewatered cakes is 72 to 77%. If the moisture content is less than 75%, the incineration efficiency is satisfactory and the consumption of supplementary fuel is comparatively small. If the moisture content is over 75%, it is very hard to keep the incinerator at the proper temperature. Especially the low temperature at the third and fourth stages, pre-heating stages, causes imperfect combustion. Hence poor incineration efficiency.

For the maintenance of incinerators, wear and corrosion of fans, rabble arms and rabble teeth in the furnace and damage of bricks at the hearth have to be checked from time to time.

4.4 SOME CONSIDERATIONS ON TOXIC SUBSTANCES AND HEAVY METALS IN DISPOSING SLUDGE

As of 1975 approximately 400 tons of dewatered cakes is incinerated and 50 tons of ash has to be disposed of per day. As the sewerage system of the city is to be expanded, a remarkable increase in quantity of sludge and ash is anticipated. Consequently in the sludge incineration and ultimate disposal, we have to use every discretion for behavior of the toxic substances and heavy metals contained in the sludge and ash.

As shown by Table 4.6, the concentrations of toxic substances and heavy metal in the inflowing sewage and the effluent are very low. However, they tend to be adsorbed and concentrated in the sludge. Therefore, it has to be carefully examined whether they pollute the air or not when incinerated and whether they cause pollution of ground water or not after the ash is used for landfill.

4.4.1 TOXIC SUBSTANCES IN THE EXHAUST GAS FROM INCINERATORS AND THE COUNTERMEASURES

The concentrations of the toxic substances and heavy metals in the dewatered cakes are not uniform. An example of analytical data is shown in Table 4.6. The concentration of iron is especially high, because ferric chloride is added for the coagulation.

The exhaust gas from the incinerators is controlled by laws and regulations. That is, the air pollution prevention law and the public nuisance prevention regulation of Kyoto Prefecture regulate particulate matter and sulfur oxides. The offensive odor prevention law restrains nasty smell, such as ammonia, hydrogen sulfide, and trimethylamine.

The exhaust gas is treated by the scrubber of wet cyclone type. The analysis table of the exhaust gas before and after the scrubber is Table 4.7. The concentrations of the particulate matter, hydrogen sulfide and trimethylamine at the outlet of the scrubber are within the standards.

No laws nor regulations control the toxicants and heavy metals from the sludge incinerators. However, compared with the standards set for other incinerators, some of the figures of the sludge incinerators are beyond the standards. It means that the treatment by the single stage scrubber is not satisfactory.

Therefore, remodeling of the incinerators is now being considered for the perfect treatment of exhaust gas. According to the plan, a cooling tower with two stage elutriation system, a tower for the elimination by means of sodium haydroxide solution and ferrous sulfate solution, and an electric dust collector are to be installed next to the existing scrubber so that particulate matter, sulfur oxides, nasty smell, toxic substances, and heavy metals as well as white smoke may be eliminated more effectively.

4.4.2 EFFLUENCE TEST OF TOXIC SUBSTANCES AND OTHERS FROM ASH

The concentrations of toxic materials and heavy metals in the ash are shown

Table 4.6 Toxic Substances and Heavy Metals in Various Samples

(ppm)

	Inflowing Sewage			Effluent			Dewatered Cake		Ash	Ash Solution*			
	Toba	Kisshoin	Fushimi	Toba	Kisshoin	Fushimi	Toba	Fushimi		Aver.	Max.	Min.	Standards (less than)
Mercury	0.0020	0.0017	0.0003	0.0001	0.0003	0.0001	5.25	2.22	0.077	0.0000	0.0000	0.0000	0.005
Cadmium	0.000	0.000	0.000	0.000	0.000	0.000	4.72	1.82	3.7	0.000	0.000	0.000	0.3
Lead	0.00	0.00	0.00	0.00	0.00	0.00	105	160	20.4	0.04	0.22	0.00	3
Organic Phosphorus	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		0.00	0.00	0.00	1
Hexavalent Chromium	0.00	0.00	0.00	0.00	0.00	0.00				0.05	0.42	0.00	1.5
Arsenic	0.001	0.005	0.001	0.001	0.002	0.001	9.73	10.10	14.4	0.011	0.053	0.000	1.5
Cyanide	0.06	0.04	0.00	0.01	0.02	0.01	0.51	0.15	0.72	0.01	0.02	0.00	1
Alkylmercury	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000							0.0005
Copper	0.15	0.23	0.03	0.01	0.06	0.01	956	250	897	0.03	0.14	0.00	
Iron	0.40	0.34	0.24	0.03	0.12	0.03	46,800	24.500	52,600	1.11	6.05	0.00	
Zinc	0.54	0.42	0.09	0.07	0.13	0.05	1,850	780	2,921	0.09	0.80	0.00	
Manganese	0.05	0.09	0.05	0.04	0.06	0.00	486	152	1,035	0.01	0.03	0.00	
Nickel	0.05	0.03	0.00	0.01	0.01	0.00	138	48	250	0.01	0.05	0.00	
Chromium	0.07	0.06	0.00	0.01	0.02	0.00	497	65	548	0.45	1.90	0.00	

* The sample is the 10% (w/v) ash solution filtered by Grade No. 5C filter paper after it had been constantly shaken for six hours.

Table 4.7 Analysis of Exhaust Gas

		Inlet of Scrubber	Outlet of Scrubber	Standards (less than)	Remarks
Particulate Matter	g/Nm³	1.00	0.20	0.7	Air Poll, Prev. Law Pub. Nuis. Prev. Reg. of Kyoto Pref.
Sulfur Oxides	Nm^3/hr	3.94	1.74	6.54	
Ammonia	"	0.004>	0.004>	134	Offensive Odor Prevention Law
Hydrogen Sulfide	"	0.155	0.137	2.685	
Trimethylamine	"	0.0023	0.0015	0.671	
Methyl-mercaptane	"				
Methylsulfide	ppm	0.12	0.24		
Mercury	mg/m^3	0.45	0.27		Standards set by Public Nuisance Prevention Regulations of Kyoto Pref. to be applied Incinerators of Wastes, such as Waste Oil and Rubber.
Cadmium	"	0.28	0.13	0.3	
Lead	"	2.89	2.70	0.5	
Arsenic	"	0.013	0.01>		
Cyanide	"	13.4	29.0	40	
Copper	"	1.1	0.28	0.3	
Zinc	"	12.22	3.42		
Nickel	"	1.1	0.29	3	
Chromic Acid	"	0.47	0.21	0.3	
Hydrogen Chloride	ppm	9.4	17.2	20	
Chlorine	"	7.9	8.0	3	

in Table 4.6. When the ash is used for landfill, a caution must be taken against pollution of ground water by them. If the concentrations of a toxic substances in the solution, which is made with the ash in accordance with the regulation, exceed the standards, the sludge is adjudged toxic and has to be disposed of by the method specially stipulated.

For the confirmation of freedom from toxicity, the solution of ash is regularly tested. The results of recent ten tests are summarized in Table 4.6.

Only a few toxic substances have been found by the tests, but the concentrations are much less than the stipulated standards.

Thus there is no problem of toxic substances. However, calcium and some others are found in the ash solution as shown in Table 4.8 and the pH of the solution is alkaline. Figure 4.6 shows the relationship between the pH in the solution and the dilution ratio of the stipulated solution diluted with ground water.

Table 4.8 Composition of Ash Solution

pH		10.64
Total Solids	mg/l	2,098
Total Hardness	,,	1,154
Calcium Hardness	,,	841
Magnesium Hardness	,,	313
Chlorine Ion	,,	253.0
Alkalinity	,,	92.3
Silica	,,	27.5

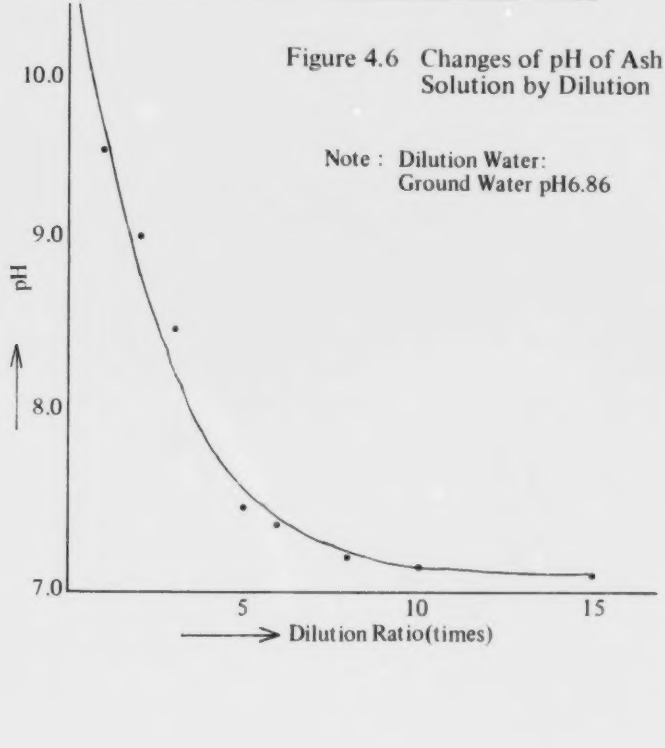


Figure 4.6 Changes of pH of Ash Solution by Dilution

Note : Dilution Water:
Ground Water pH6.86

4.5 RESEARCHES ON SLUDGE ASH FOR ITS REUSE

It is expected that the quantity of sludge ash will rapidly increase from now on. However, it is very hard to obtain the space for filling up. Therefore, it is necessary to develop a new method to effectively reutilize the ash. Now researches are under way on such matters as how to revivify the ash in nature, how to use the ash as a material for road construction, how to use the ash to make a lightweight aggregate, and others.

4.5.1 UTILIZATION FOR SOIL CONDITIONERS

Table 4.9 shows an example of the fertilizer components in the ash. The slaked lime that is added as a coagulant at the dewatering process remains as residue in the ash. Therefore, it can be used as a neutralizer of acid soil. Further it can be used as a fertilizer because of the components shown in the Table 4.9. Experimentally the ash is effective for the growth of root-crops. The growth of aquatiic rice is also satisfactory if it is given appropriately. However, more careful researches are required on the effect of the heavy metals contained in the ash before it is widely used as fertilizer.

4.5.2 USE AS A MATERIAL FOR ROAD CONSTRUCTION

Experiments prove the possibility of using the ash as the material of subgrade and subbase course of the road. In October 1973 when a road was constructed in the Toba Plant, the ash was tentatively used as the material of the subgrade. As for the quality the result seems satisfactory, though it is still being carefully observed. To fill an excavated area in the Plant the ash was also used.

As shown by Table 4.10, the particles of the ash are very fine. Therefore, when it is used as a construction material, a caution must be exercised not to scatter it.

Table 4.9 Fertilizer Components of Ash

(%)

Components	
N	0.10
P_2O_5	4.00
K_2O	0.35
Fe_2O_3	11.39
SiO_2	29.91
CaO	34.22
MgO	2.55
MnO	0.14
B_2O_2	0.05

Table 4.10 Grain Size Distribution of Ash

(%)

Grain Size (mm)	
4.76 - 2	2
2 - 0.42	18
0.42 - 0.074	27
0.074 - 0.005	48
Less Than 0.005	5

CHAPTER 5. SLUDGE PRODUCTION AND SOLID LOADING BALANCE IN THE NISHIYAMA STP, NAGOYA

Outline of the Nishiyama STP 47
Equipment and Devices Installed 47
Primary Sludge Production 48
Sludge Production through the Actuated Sludge Process 49
Increase in Sludge Production by Chemical Addition 49
Summary and Future Tasks 49

5. SLUDGE PRODUCTION AND SOLID LOADING BALANCE IN THE NISHIYAMA STP, NAGOYA

Methods for ultimate disposal and sites for disposal of sludge are limited in Japan. Therefore, to obtain correct information on sludge production and its quality in sewage treatment plants is of vital importance to setting up future measures for treatment and disposal of sludge.

As people's life style, especially their dietary life, changes year by year, sludge production is increasing in sewage treatment plants. Also, the quality of sludge is expected to change: for example, its organic content will increase.

The purpose of the project is to obtain information on sludge production, quality changes, kinetics of sludge production, etc. by means of a long-range measurement at a typical treatment plant of domestic sewage which introduces the activated sludge process. The Nishiyama STP in Nagoya has been selected and surveys have been carried out jointly by the Nagoya City and the Public Works Research Institute, Ministry of Construction. All data collected during two years of surveys will be finally analyzed by a statistic measure.

Discussed in the following are equipment and devices installed in the Nishiyama STP for these surveys and results of the survey of its beginning stage.

Outline of the Nishiyama STP

In the Nishiyama STP, sewage is collected by separate sewers, and biological treatment by the conventional activated sludge process is conducted after the primary treatment. Population served is 46,000 and the average flow is 20,000 m^3/day (5 mgd).

Raw sewage quality is as follows (daily average of the period from February to June, 1975): BOD_5, 70 ~ 125 mg/*l* (average 100 mg/*l*) and SS, 60 ~ 230 mg/*l* (average 130 mg/*l*). Diurnal variation is large and thus STP is a typical treatment plant for domestic sewage. Average loadings per capita base computed from average influent polutant loadings, 46 g/person/day for BOD_5, 60 g/person/day for SS, 10.9 g/person/day for TN, and 1.52 g/person/day for T-P, all of which are average values in Japan.

Fig. 5.1 shows the flow diagram of the STP. Its bays are separated into two, to one of whose aeration tank alum is added now.

Sludge treatment is not carried out in the STP. Produced sludge is stored in holding tanks, then pumped to nearby sludge treatment plants periodically. Because supernatant is not returned from sludge loading facilities, the STP is convenient to obtain exact loading balance.

Equipment and devices installed

In this STP, withdrawal of sludge from the primary setter has been intermittently performed 4 to 5 times/day and that of waste activated sludge, 1 to 2 times/day, depending on its production.

Since it is difficult to measure manually all of the sludge, three electromagnetic flow meters for measuring sludge flow rate and three ultrasonic solidmeters for measuring solid in the sludge have been installed. Fig. 5.1 shows their respective locations. Of the three, one is for measuring sludge withdrawn from the primary setter while the other two are for measuring waste activated sludge. Values measured are transmitted to the control room and continuously recorded there. Fig. 5.2 shows installed meters. During four months until July, 1975, major troubles have not occurred in connection with these meters.

Observations from the solidmeters remain stable for waste activated sludge concentration and the differences with observed values are within 10%. This may be because in the case of waste activated sludge, fluctuations of concentration is small, its quality is stable, and its withdrawal is conducted in a comparatively longer period (about 30 minutes) with a pump. On the contrary, since sludge in the primary setter is withdrawn in a shorter period (about 5 minutes) by gravity and its concentration changes greatly, it was difficult to get accurate results.

To know the potentials for sludge production, it is necessary to continuously measure the water quality of influent and effluent of each facility. To measure the concentration of suspended solids and organic matters continuously, turbidimeters and TOC meters are scheduled to be install.

Because corelation between turbidity and SS has been found, measured turbidity is converted into SS values. As turbidimeters, the surface scatter type, and as TOC meters, the type which continuous sample dose is possible, will be introduced. Fig. 5.1 shows locations of turbidimeters and TOC meters. As for TOC, it is planned to measure samples from each location after dividing them into "total" and "soluble."

Primary sludge production

Table 5.1 shows the analytical values of the primary sludge. The percentage of organic matters in sludge amounts to as high as 85%. The ratio of TKN and T-P in the total solids was 1.7 to 3.7% and 0.3 to 0.7%, respectively.

Primary sludge production is the balance between solid amount in influent and that in effluent. Until turbidimeters were installed, SS had been analyzed using composite samples for 24 hours. Fig. 5.3 shows the relations between observed values of solid quantity of influent and effluent and primary sludge withdrawn obtained from values shown by flowmeters and solidmeters. Data are those from April to mid-June. Figures in the upper column are the observed values of withdrawn amount and those in the lower column are quantity of influent and effluent solids, the balance of these two being estimated values of withdrawn amount.

As shown in the Figure, values measured by meters and those obtained from the balances between influent and effluent quantity are not very well correspondent with each other.

Reasons for this disagreement are not very clear but causes which can be considered are: (1) the composite sample used was not representative; (2) mistakes or errors in calibrations of meters; and (3) analysis method of SS cannot represent the total quantity of the primary sludge.

Table 5.2 shows the operational condition of the primary setter, and Table 5.3 shows average water quality.

Sludge production through the activated sludge process

It has been pointed out that production of waste activated sludge is influenced by a lot of factors. At this moment only a few data have been obtained, so discussions on kinetics of production is not done here.

Fig. 5.4 shows reletions between waste activated sludge produced in the conventional activated sludge process and solids in influent and effluent. The former was obtained from flowmeters and solidmeters while the latter was gained from composite samples for 24 hours, both manually. When MLSS in the aeration tank is constant, the net sludge production is obtained by substructing influent solids from the total of waste activated sludge and effluent solid.

During this period, average daily influent BOD_5 loading was 680 kg/day, average daily effluent BOD_5 loading, 105 kg/day, and removed BOD_5 loading, 575 kg/day. Other operational conditions are shown in Table 5.2 and 5.3.

Tranfer rates into sludge per day computed based on soluble BOD_5 vary widely. After TOC meters are installed, studies on transfer rates based on TOC, in addition to BOD_5, and the production kinetics will be conducted using measurement data covering a long period.

Increase in sludge production by chemical addition

Production of waste activated sludge at the bay where alum is dosed at the extreme end of the aeration tank has been measured.

Alum is dosed so that Al/P may become 2 in the mole ratio. $Al_2(SO_4)_3H_2O$ is the ingredient of alum and the concentration of dose is 8 mg Al/P.

Because the bay where alum is dosed and that where it is not dosed have the same influent quantity, production of them have been compared on the basis of data from May to June, 1975. The results are shown in Table 5.4.

The quantity of sludge at the bay with alum has increased more than that at the bay without alum. The increase rate was nearly 40% for the former.

In the Table, the estimated values of the increase in terms of aluminum hydroxide and aluminum phosphate are shown. From these values, it is known that the quantity of alum dosage equals the increase in production of waste activated sludge.

Comparison between them are planned to be examined by changing the dose rates of alum.

Summary and future tasks

The outline of the project at the Nishiyama STP to make a survey on sludge production from sewage treatment plants started in February, 1975 has been described in the above. Because this project is still at its initial stage, sufficient information has not been obtained yet. It is expected, however, that the results to be gained from this project will give data useful to sludge treatment and disposal plans at sewage treatment plants to be built in the future.

Future tasks of the survey of the project will be as follows:

1) By observation of sludge production for a long period of time, to clarify seasonal fluctuations of its quantity and quality, yearly changes, etc.

2) To install turbidimeters and TOC meter and to clarify effects of concentration of suspended solids in sewage on sludge quantity and the kinetics of transfer into sludge in the process of biological process of organic matters.

3) To clarify material loading balance in sewage treatment plants.

4) To study a long-range view for methods of sludge treatment and disposal at sewage treatment plants, mainly on the treatment of domestic sewage.

Table 5.1 Primary Sludge Characteristics

Item / Sample	V.S content (%)	TKN content		T-P content	
		TS base (%)	VS base (%)	TS base (%)	VS base (%)
1	83.4	1.2	1.5	0.3	0.4
2	85.7	3.1	3.7	0.7	0.8
3	84.7	1.4	1.7	0.7	0.4

Table 5.2 Operational Conditions of the Plant

Average daily flow (m^3/d)	Primary settler		Flerator					Final settler	
	Detention time (hr)	Overflow rate ($m^3/d/m^2$)	Filtration time (hr)	MLSS (mg/ℓ)	MLVSS (mg/ℓ)	SVI	Organic load (kgBOD /kg SS)	Detention time (hr)	Overflow rate ($m^3/d/m^2$)
21,300	1.8	38	4.1	1,100	720	105	0.36	3.1	24

Table 5.3 Waste Water Characteristics

(mg/ℓ)

Category / Item	Raw sewage		Primary eff.		Final eff.	
	Ave.	Range	Ave.	Range	Ave.	Range
Turbidity	80	56 ~132	52	32 ~92	6.7	4.1 ~11.3
S.S	118	81 ~162	41	31 ~78	6	2 ~11
BOD_5	99	69 ~124	64	47 ~95	9.5	6.5 ~16.5
TOC	80	51 ~122	59	38 ~87	19.3	8.3 ~14.5
T-P	3.54	2.59~ 4.92	3.45	2.01~ 4.40	1.25	0.59~ 1.97
TKN	25.2	21.8 ~ 29.1	25.8	20.6 ~39.1	13.1	9.9 ~21.0

Table 5.4 Comparison of Sludge Production

Sludges	Control	Alum addition
Waste activated suldge		
kg/d	536	545
(mg/ℓ)**	(45.4)	(46.1)
Solid in effluent		
kg/d	77	310
(mg/ℓ)**	(6.5)	(26.2)
Total		
kg/d	613	856
(mg/ℓ)**	(45.4)	(72.5)
Addition as $Al(OH)_3$*		
kg/d	–	184
(mg/ℓ)**	–	(15.6)
Addition as $AlPO_4$*		
kg/d	–	139
(mg/ℓ)**	–	(11.8)

* Calculated value. ** Based on inflow.

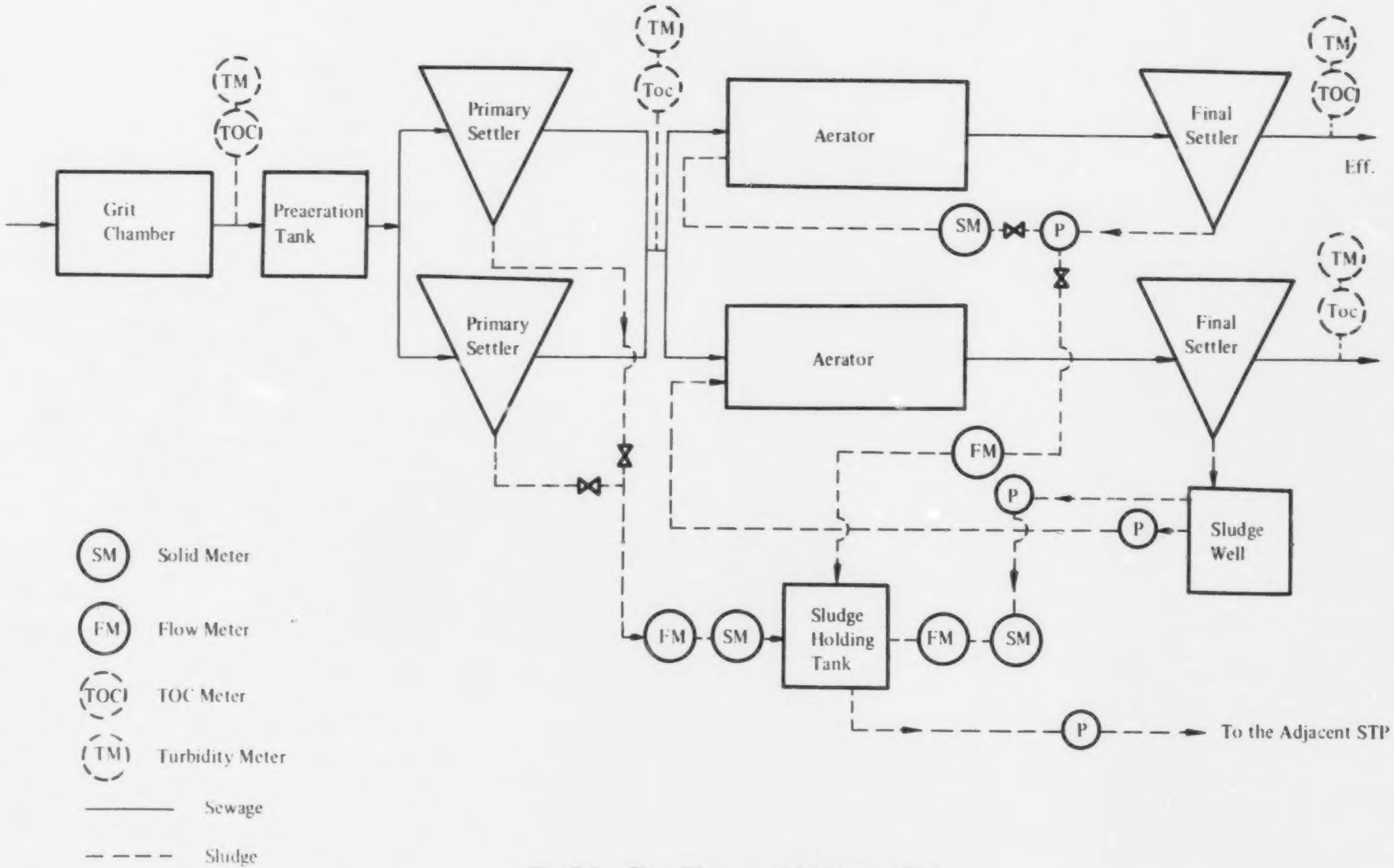

TM
TOC
Grit Chamber
Preaeration Tank
Primary Settler
Primary Settler
TM
Toc
Aerator
Aerator
Final Settler
Final Settler
TM
TOC
Eff.
TM
Toc
SM
P
P
FM
P
P
Sludge Well
FM
SM
Sludge Holding Tank
FM
SM
P
To the Adjacent STP
SM
Solid Meter
FM
Flow Meter
TOC
TOC Meter
TM
Turbidity Meter
Sewage
Sludge

Fig. 5.1 Flow Diagram of Nishiyama STP

ILLUSTRATION NOT AVAILABLE

Fig. 5.2 Installed Flow Meter and Solid Meter (Waste Activated Sludge Pipe)

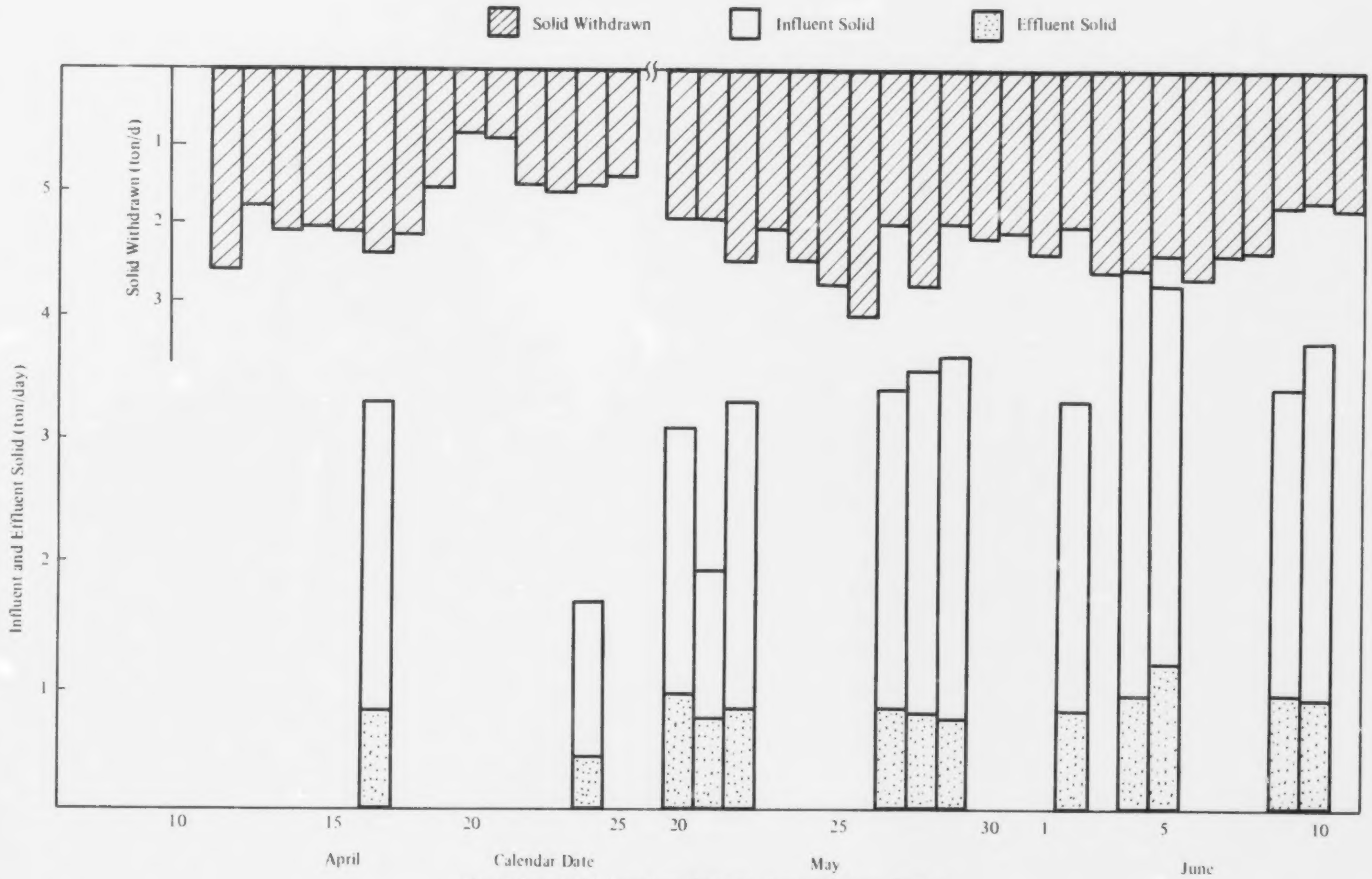


Fig. 5.3 Solid Loading Balance Over Primary Settler

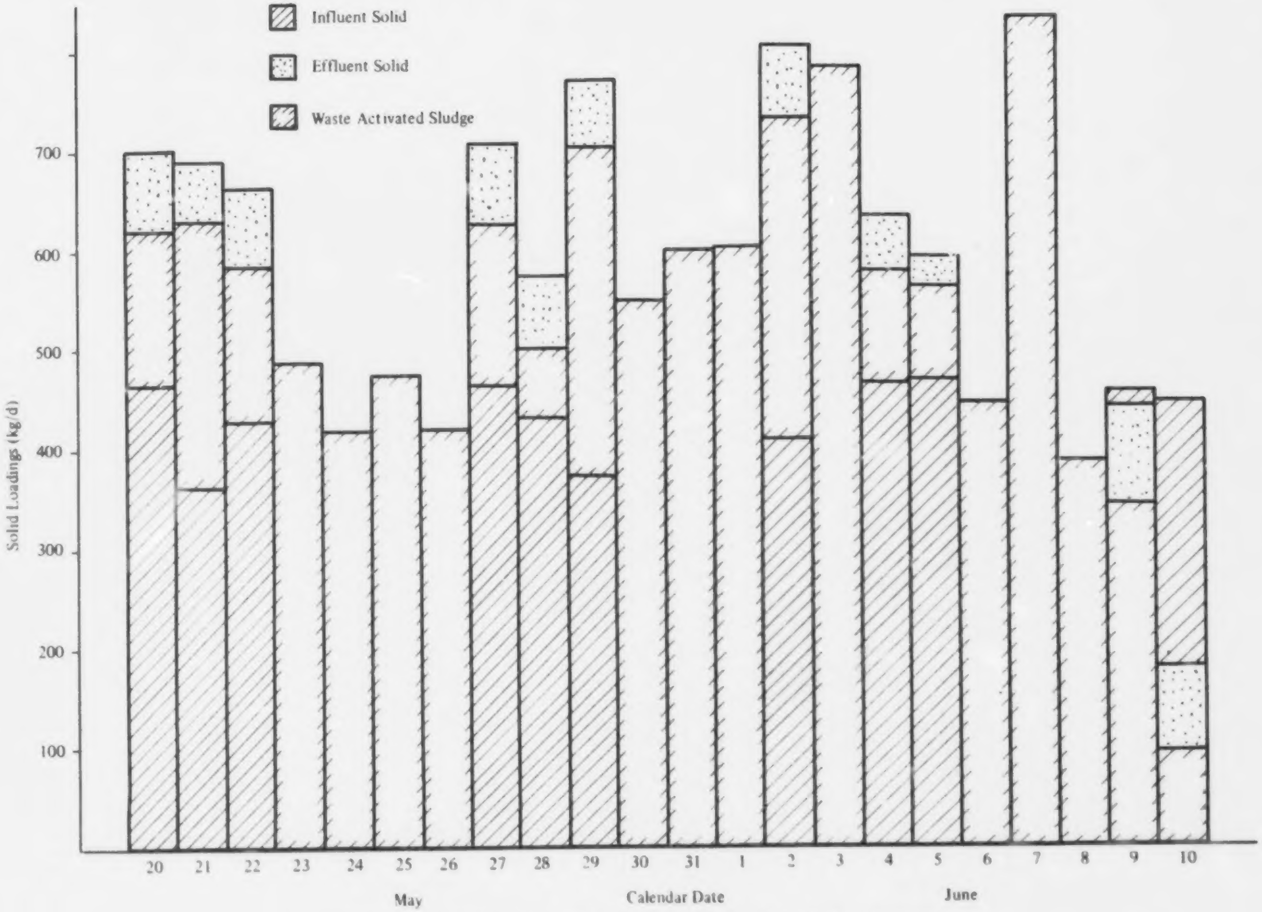


Fig. 5.4 Solid Loading Balance Over Activated Sludge Process

Fourth US/JAPAN Conference
on
Sewage Treatment Technology
Paper No. 2

AUTOMATION AND INSTRUMENTATION FOR WASTEWATER TREATMENT PLANT

October 24, 1975
Cincinnati, Ohio

Ministry of Construction
Japanese Government

AUTOMATION AND INSTRUMENTATION FOR WASTEWATER TREATMENT PLANT

1. **Basic Conception of Automation and Instrumentation in Sewage Treatment Plant** 61
A. Sugiki, Japan Sewage Works Agency

2. **Automatic Water Quality Measurement for Wastewater Treatment** 72
K. Murakami, PWRI, Ministry of Construction

CHAPTER 1 BASIC CONCEPTION OF AUTOMATION AND INSTRUMENTATION IN SEWAGE TREATMENT PLANTS

1.1 Introduction 62
1.2 Some Points to be Solved in Automatic Control 62
1.3 Basic Conception for Automatic Control 63
1.4 Profile of Automatic Control Systems 64
1.5 Future Research 67

1.1 INTRODUCTION

Several different characteristics exist in the waste water treatment process as compared to such process at an ordinary manufacturing process. These particularities are necessary to be taken into consideration when the instrumentation and the automation are introduced into a sewage treatment plant. For instance:

(a) Both quantity and quality of sewage flow vary to a great extent regardless of the optimum conditions of a sewage treatment processes. Accordingly, it is of necessity to forecast and preestimate it to some extent for operation as optimum.

(b) Since a sewage treatment plant concerns itself widely in the public interests and the effluent therefrom has a great impact to the environment, it is needed always to secure stable and best water quality of effluent. Minimum required conditions, of course, should be maintained even in an emergency.

(c) Time required for through whole treatment process after arriving at a treatment plant ranges from a few hours to 10 hours. In other words, the time constant is considerable long.
Therefore, in order to increase the treatment efficiency the feed forward control is requisite.

(d) Although mechanical and physical treatment process such as screening and sedimentation are included in the waste water treatment process, biological treatment plays a main role for waste water treatment in Japan. Hence a control system should be built up for the most suitable environment to microorganism.

As the sewage treatment plant becomes larger, process has rather long time constant and combination of complicated many different systems, so it is very difficult to operate it properly. To cope with this complexity it has been necessitated to introduce the technique of electronic instrumentation as to systematic control on the process.

Types of instrumentation and control devices range from a analog controller to a large-scale digital computer, and all are being in wide use according to the purpose of control and the scale of treatment plants. As to introducing a computer although there are arguments the general trend goes to the introduction of computer. The case of waterworks which is rather ahead in the instrumentation and the automation of water purification plants gives a good example. At the present, the use of computer is limited to a role of an information center: such as data collection, indication, recording, supervision, data storage and calculation. However, with the further progress of studies on the mode of biological treatment process and development of software, various types of feed forward control, adaptive control or optimizing control will be made available, and the function of computer control may be far be improved.

1.2 SOME POINTS TO BE SOLVED IN AUTOMATIC CONTROL

If the technique of automatic control is introduced into the operation and maintenance of a sewage treatment plant, it will become possible to control more effectively for treatment process under the characteristics of the sewage treatment mentioned above, and at the same time the reduction of personnel expenses and further more than cutback of treatment cost and also the improvement of working conditions of the staff to be engaged in as well as the labor and skill saving are expected. However, the relation between the effects or benefits of the automatic control and the costs required for it, that is to say, the cost performance, would be variable according to the scale of treatment plants and/or the respective local conditions such as fluctuation of water quantity and quality. There are some technical and operational problems left to the future to need technological development as follows.

(a) Highly dependable various automatic measuring devices and instruments are required for the automatic control. Many of the quantitative measuring instrument such as a flow measuring devices and a water level meter are available now, except some modification in needed some cases. On the contrary, the insufficient performance of reliability, responsibility, stability and repeatability etc. particularly of detectors (sensors) of the qualitative measuring instrument such as a suspended solid or an organic substance, BOD, COD, TOC etc., has been pointed out.

(b) Operation and maintenance of treatment plant usually depend on the experience and intuition of skilled employees. When the automatic control is put into practice, it becomes necessary to select suitable operational indices for this kind control. We need kinetic model of treatment process, and now strongly conducted studies but unfortunatedly not yet the models are determined. Further development of measuring apparatus as well as kinetic models of treatment process would be expected.

(c) With the implementation of the automatic control, manner of operation and maintenance are conducted in different ways. Therefore, it is highly necessary to establish a system of education and training for the staffs in charge so that they may fully understand the system composition and how it works and may perform a proper operation and they may take necessary action promptly and correctly to cope with abnormal condition in emergency.

1.3 BASIC CONCEPTION FOR AUTOMATIC CONTROL

The basic conception for the water treatment system was formulated as follows after having been taken into consideration the previously-mentioned characteristics of the process at this time.

(a) Since the studies of process kinetics and the development of respective water quality sensors and still incomplete at the moment, it is appropriate to lay more emphasis on the quantitative control. These points, however, seem to be developed step by step with time. After evaluation on new developments, new techniques are introduced for the qualitative control time to time.

(b) Although sewage treatment plants are mostly planned to complete with in next 20 years, a stage construction method is usually applied. Therefore, the automatic control should be so designed as to function at every stage respectively. For the treatment plant, of which designed capacity exceeds 100,000 cubic meter employed, a computer should be introduced.

(c) The quantitative control plays a main part presently, but as treatment plant become larger in scale, items of supervision and data to be handled increases so much that the operation under man power may exceed his ability. Hence, a computer should be introduced to ensure the more effective and more detailed automatic supervision on various apparatus in plant to keep operation record and process control. In this case, back up system should be prepared in order to secure to keep good operation in case of emergency.

(d) Pumping stations for sanitary sewage and storm sewage need to be so designed that they will be operated automatically according to the scale, rather small and at the same time, it is necessary to ensure the remote supervision and control from a treatment plant or the nearest pumping station.

(e) In future, it is required to measure rainfall precipitation, water level, amount of flow and water quality, etc. at the desired points in the catchment area and to transmit to a central control office through telemeter devices so as to enable more reliable rational operation to control stormwater.

(f) Even though a treatment plant is planned with quantitative automatic control, the necessary incidental facilities are needed at the planning stage so that the qualitative automatic control will be possible in future.

1.4 PROFILE OF AUTOMATIC CONTROL SYSTEMS

Sewage treatment process consists of pumping station, preliminary sedimentation tank and chlorination tank all working as one system.

Explanation will be made below as to the forms and methods of control, while referring to those machinery, instruments and devices that are included in the controlled system by each establishment. It is briefly described on the control system in each process.

(a) Pumping station

Pumping stations are cassified into booster pumping station and those in the treatment plants, and also collecting sewage system differ from the separate system to the combined one. Fundamentally for all the types control systems of pumping station are about same one with little alternation by the case. Apparatus such as inlet gate, screening, grit chamber, sanitary sewage pump and storm sewage pump are to be controlled in a pumping station. Inlet gates are manipulated so as to control incoming sewage to treatment plants. Operation of the gate are controlled by sign of the unusual rise of water level at an inlet or at a wet wall. Usually, when water level goes over the height already set up, the gate will be closed automatically with a use of analog regulators, etc., and the manipulation to open it is performed at on site or through the remote manual operation if possible, which needs such as reservoir pond. Of course, in case the incoming flow rate is desired to be controlled for the better performance of treatment, it should be designed to make possible the closing gate through the remote manual operation. The control in this case will be needed such devices that the sequence control device coupled with the measuring apparatus are to be required. In future, if incoming flow rate of sewage can be forecast with a use of rainfall precipitation data in a catchment area and so on, or the forecast of the quality of incoming water will be made available with a use of monitoring apparatus of water qualities, the feed forwared control and adaptive control become applicable with the additional introduce of the processer control apparatus.

If there are installed grit chamber, the adaptive control and the optimizing control should be applied thereto with the surface loading or the velocity of flow in the chamber being constant rate. In the control with a constant rate of surface loading, an inlet gate is manipulated after selecting the number of chamber according to the incoming flow rate of water. The control with a constant velocity of flow in the chamber is performed by manipulation an inlet gate and an outlet gate after selecting the number of basin according to the incoming flow rate and the water depth of the chamber. In this case, the sequence control device, the measuring apparatus and the process control apparatus are necessary and the control is conducted by sequential computer control method. As for grit chamber, the programmed and the conditional control are both used jointly, and in the former the time scheduled operation is performed with a use of incoming flow pattern, while in the latter it works only when a difference in water level before and behind a screen exceed a fixed difference. A screen scraper and lifting machine and a belt conveyer are naturally in combinated, and the control appratus are the sequence control device and the measuring appratus. Further, a control based on the forecast of the volume of screenings produced will be installed in future. Since it seems possible to presume the volume of screenings produced on the total volume of incoming flow, the processer control appratus is necessary to be set up so that equipment of screen, i.e. rake scrapers may start working when the total volume reaches a fixed amount.

In respect of grit removal facility, the control method is also the same as that of a screening devices. The programmed control, under which the time scheduled operation is performed with a use of incoming flow pattern, is applied to start working machine at the condition which the facility starts working when the volume of grit deposit reaches a fixed height.

Automatically-operated sanitary sewage and storm sewage pumps are to be under the level control (conditional control), and thereby they start working when the water level at wet well (in the case of a storm sewage pump, the water level reaches a fixed level which pump should be started) reaches the upper limit of a fixed point and stop when reaching the lower limit.

In the case of a sanitary sewage pump the level control depending on the water level at a wet well for rather unusual condition, flow control are needed so that inflow and outflow are regulated to maintain as to the certain flow velocity in grit chamber as possible. The outflow rate will be controlled through the selection of the number of pump, the number of revolution and the opening of control valve, and the sequence control device and the measuring apparatus for are required for this purpose. If the optimum control of outflow rate is planned in future, the processer control apparatos will become essential.

A storm sewage pump is controlled depending on the condition based on the water level at wet well or at the point where effluent discharge. In future, if the time comes when the inflow pattern, the detention time of sanitary sewer, etc. can be forecast through the rainfall, or when the necessary data for estimate incoming flow rate of sewage can be transmitted from a booster pumping station and/or the desired points along a force main just like in the case of a storm sewage pumping station in the treatment plant, the feed forward control will be made available by adding the processer control apparatus.

(b) Primary sedimentation tank

Such appratarus as sludge pump, a withdrawal valve and a scum collecting device are to be controlled by instrumentation. A sludge pump and a withdrawal valve start working by signal of timer and stop when sludge density lowers down to desired concentration or when the sludge volume withdrawal reaches are fixed volume. In future, it is necessary to combine the level control under which they start working when the measured volume of sludge deposit reaches the upper limit of a certain height and stop when sludge height reaches the lower limit.

A scum collecting device is switched on and off intermittently at on site or through the remote manual-operation, aside from being controlled by signal of timer. In future, the feed forward control is to be introduced depending on pattern of scum volume produced which is forecast by incoming flow rate, water quality. Accordingly, the processor control apparatus will be needed in future in addition to the sequence control device and the measuring apparatus.

(c) Aeration tank

Appratus that are the controlled system of an aeration tank include a blower, a suction valve for air supply, and a return sludge pump. A blower and a suction valve are to supply an adequate volume of air for the microbic reaction in an aeration tank. There are two kinds of control systems on dissolved oxygen in aeration tank. One is the proportional control of incoming flow rate of sewage and the other is to keep constant dissolved oxygen concentration in the tank. The former is the control in supplying the air in proportion to the incoming flow rate of sewage, whereas the latter is to control the air by signal from a dissolved oxygen meter so as to maintain the dissolved oxygen concentration in an aeration tank at a fixed concentration. The air supplied is adjusted by the number of blower and the opening of suction valve. In future, the optimum control is to be employed which are depending on the estimation of the required volume of air from the incoming flow rate and water qualities (BOD, COD, TOC etc.) as well as of signal from a dissolved oxygen meter in aeration tank. In the case of a return sludge pump, the number of pump and the number of revolution are controllable, based on the required volume of return sludge. As for the switchover of the number of pump, a starting order would be arranged so as to the same running period of respective pumps. Its control method include the proportional control of the incoming flow rate and the MLSS at the constant concentration based on the density of return sludge. In this cases, it is necessary to equip the sludge holding tank for preperly return sludge. In future, the optimizing control should be performed in consideration of the biological activate of return sludge as well as the BOD load (f/M) after measuring the BOD (COD, TOC) in the raw sewage.

(d) Final sedimentation tank

In the final sedimentation tank a excess sludge withdrawal pump which starts working under signal of a timer and stops when the sludge density lowers down to a fixed concentration, or when the top of sludge zone reaches down to a fixed height. The timer is being set itself upon the programmed control utilizing a pattern of incoming sewage. In future, the level control will be put to use under which a excess sludge withdrawal pump starts working when the height of sludge deposit reaches the upper limit of a fixed height and stops upon its down to the lower limit. Further, it is to focus the optimizing control of sludge distribution of tanks by estimation of sludge produced from the quality of raw sewage for example the BOD or COD, TOC.

(e) Chlorination facility

There is some difficult situation whether chlorination of sewage effluent is effective means or not especially regards to downstream water supply system. At present time, chlorination is required to sewage effluent if coliform counts in the effluent exceeds 3000/ml. Next action not yet determined in relation to chlorination, but some alternation will be expected.

Chlorination is applied in proportion to the flow rate of effluent, but its concentration of chlorination in effluent would be determined measuring the concentration of residual chlorine and the counts of coliforms.

In future, as the optimum control of chlorination would be such manner as injected after measuring automatically the residual chlorine, then the processer control apparatus is to be required in addition to the sequence control device and the measuring and operational apparatus.

A detoxitation devices are required for emergency emission of chlorine gas in chlorination chamber. An aspiration blower of the escaping gas and an injecation pump of the dechlorination agent such as sodium thiosulfate start working by signal from detector for chlorine gas. The manipulation to stop of dechlorination devices should be done at the machine side for the sake of safety.

1.5 FUTURE RESEARCH

The problems on the development of automatic control of treatment plant operation were pointed out, and the best practical system at present was proposed.

Also target system in future is shown, but joint efforts of government people and manufacturers is inevitable for the application.

Study report about automatic control development has been presented since 1960 in Japan and even simulation model study by computer is published now.

Since 1974 development study by a manufacturer granted from Ministry of Construction and same study by Japan Sewage Works Agency started from investigation of many types of sensor and process kinetics Analysis.

The abstract of our study objects is as follows.

(a) We are going to do systematic investigation of durability, operational difficulty and accuracy of measuring equipment and make the questions clear.

(b) We are going to design and construct treatment plant which adops best applicable full automatic controll system proposed by us. Then the cost-effectiveness performance will be cleared.

(c) We are continueing the kinetics study of actual plant operation. And using 30 m^3/day pilot plant which will be complete by 1976, simulation experiment is going to be done under various operation conditions to aim quality control application in future.

Items To Be Measured Manually at a Final Sedimentation Basin

(1) Cadmium and its chemical compound;
(2) Cyanide;
(3) Organic phosphate compound;
(4) Lead and its chemical compound;
(5) Sexivalent chrome compound;
(6) Arsenic and its chemical compound;
(7) Mercury, alkyl mercury and other mercuric compound;
(8) B O D (Biochemical Oxygen Demand);
(9) C O D (Chemical Oxygen Demand);
(10) S S (Suspended Solid);
(11) Transparency.

1. A DRAFT OF AUTOMATIC CONTROL AT SEWAGE TREATMENT PLANTS

AT THE PRESENT TIME

Water Treatment Facilities	Control Device					Items to be Supervised (Automatically)	Items for Manual Measurement	Items for Data Process	Remarks
	Controlled System	Control Form	Control Method	Constituent Apparatus	Back-up				
Pumping Station	Inlet Gate	Open Loop Control	Conditional Control (Remote Hand operation)	Sequence Control Device Processer Control Apparatus Measuring Control Apparatus	Use What is One Rank Below the Control Method	Water Level at Pump Well Water Level at Inflowing conduit		Rainfall Precipitation Water Level, inflow rate PH (Volume of screenings, Volume of Sand Removed) Volume of Sand deposit	Data Processing Device Small-scale (Punching Recorder and Integrating Meter) Medium- and Large-scale (Daily Report, Monthly Report, Trouble and Running Record using an electronic computer) Central Monitory Facility Monitoring, Operate ITV (Sand Basin, Discharging Outlet and other facilities inside a plant to be monitored)
	Screenings Facility	ditto	Conditional Control Programed Control	Sequence Control Device Measuring Control Device	ditto	Water Level Before and Behind Screen	Volume of Screenings		
	Sand Removing Facility	ditto	ditto	Sequence Control Device	ditto	(Volume of Sand Deposit)	Volume of Sand Removed		
	Sanitary Sewage and Storm Sewage Pumps	Closed Loop Control, Open Loop Control	Level Control Discharge Control Conditional Control Feed Forward Control	Sequence Control Device Measuring Control Apparatus Processer Control Apparatus	ditto	Water Level at Pump Well Inflow Rate PH			
Preliminary Sedimentation Tank	Raw Sludge Pump Pulling-out Valve	Open Loop Control	Conditional Control Programed Control	Sequence Control Device Measuring Control Apparatus	ditto	Volume of Raw Sludge Pulled out Density of Raw Sludge (Volume of Sludge Deposit)		Volume of Raw Sludge withdrawn Density of Raw Sludge Volume of Sludge Deposit	
	Scum Removing Device	ditto	Conditional Control	ditto	ditto	Volume of Wind Sent			
Aeration Tank	Blower	Open Loop Control, Closed Loop Control	Conditional Control DO Uniform Control Proportional Control of Inflowing Volume of Water	ditto	ditto	MLDO (MLSS)	SV	MLDO, MLSS (SV) Volume of Return Sludge Density of Return Sludge	Note: A storage tank for return sludge is required.
	Return Sludge Pump	Open Loop Control, Closed Loop Control	Conditional Control MLSS Uniform Control Proportional Control of Inflowing Volume of Water	ditto	ditto	Return Sludge Rate Density of Return Sludge	Residual chlorine Concentration		
Final Sedimentation Tank	Surplus Sludge Pump	Open Loop Control	Conditional Control Programed Control Processer Control	Sequence Control Device Measuring Control Apparatus Processer Control Apparatus	ditto	Volume of excess Sludge Density of excess Sludge (Volume of Sludge Deposit)	(as per attached sheet)	Volume of excess Sludge Density of excess Sludge Volume of excess Deposit	
Chlorination Facility	Chlorination Device	Closed Loop Control	Uniform Control	Sequence Control Device Measuring Control Apparatus Processer Control Apparatus	ditto	Volume of Water Discharged Chlorine dosing rate	MPN	Volume of Water Discharged Chlorine dosing rate residual Chlorine, MPN Concentration of Escaping Chlorine	
	Neutralization Device	Open Loop Control	Conditional Control	ditto	ditto	Concentration of Escaping Chlorine			

Items in the parenthesis are to be measured, but not to be used for the control

Items in the parenthesis is to be filled manually

IN THE FUTURE

Water Treatment Facilities	Control Device					Items to be Monitored (Automatically)	Items for Manual Measurement	Items for Data Process	Remarks
	Controlled System	Control Form	Control Method	Constituent Apparatus	Back up				
Pumping Station and Storm Sewage Pumps	Inlet Gate	Open Loop Control	Conditional Control Remote Hand operation	Sequence Control Device Measuring Control Apparatus Processes Control Apparatus	Use What Is One Rank Below The Control Method	Water Level at Pump Well Water Level at Inflowing conduit		Rainfall Precipitation, Water Level, inflow rate PH Volume of Sand Deposit (Volume of screenings, Volume of Sand Removed)	Data processing device is for daily report, monthly report, trouble and running record through an electronic computer. Central Monitory Facility Monitoring, Operate ITV (Sand Basin, Discharging Outlet and other facilities inside a plant to be monitored)
	Screenings Facility	ditto	Conditional Control Programed Control	ditto	ditto	Water Level Before and Behind Screen	Volume of screenings		
	Sand Removing Facility	ditto	ditto	ditto	ditto	Volume of Sand Deposit	Volume of Sand Removed		
	Sanitary Sewage and Storm Sewage Pumps	Open Loop Control, Closed Loop Control	Conditional Control, Level Control Inflowing Volume of Water Control Feed Forward Control	ditto	ditto	Water Level at Pump Well Inflow rate PH			
Preliminary Sedimentation Tank	Raw Sludge Pump Outlet Valve	Open Loop Control	Conditional Control Programed Control	ditto	ditto	Volume of Raw Sludge Pulled out Volume of Sludge Deposit Density of Sludge		Volume of Raw Sladge Pulled out Volume of Sludge Deposit Density of Sludge	
	Scum Removing Device	ditto	Conditional Control	ditto	ditto				
Aeration Tank	Blower	Open Loop Control, Closed Loop Control	Conditional Control Ratio Control of Air Volume Cascade Control Optimizing Control	ditto	ditto	airflow rate MLDO MLSS, SV		air flow rate MLDO, MLSS, SV Volume of Return Sludge Density of Return Sludge TOC (or TOD)	Note: a storage tank of return sludge is required.
	Return Sludge Pump	ditto	Conditional Control, Optimizing Control, Programed Control Cascade Control Uniform and Ratio Control of Return Sludge Volume	ditto	ditto	Volume of Return Sludge Density of Return Sludge TOC (or TOD)			
Final Sedimentation Tank	Excess Sludge Pump	Open Loop Control	Conditional Control Programed Control Processes Control	ditto	ditto	Volume of excess Sludge Density of excess Sludge Volume of Sludge Deposit	(as per attached sheet)	Volume of excess Sludge Density of excess Sludge Volume of Sludge Deposit	
Chlorination Facility	Chlorination Device	Closed Loop Control	Uniform Control of Injected Volume Ratio Control of Injected Volume	ditto	ditto	Rate of Water Discharged Chlorine dosing rate Residual Chlorine	MPN	Volume of Water Discharged Chlorine dosing rate Residual Chlorine of Escaping Chlorine	
	Neutralization Device	Open Loop Control	Conditional Control	Sequence Control Device Measuring Control Apparatus	ditto	Concentration of Escaping Chlorine			

Items in the parenthesis is to be filled manually

CHAPTER 2. AUTOMATIC WATER QUALITY MEASUREMENT FOR WASTEWATER TREATMENT

2.1 Introduction 73

2.2 Automatic Water Quality Monitoring of Raw Sewage 73
  2.2.1 Development of Automatic Cyanide Monitor 73
  2.2.2 Automatic Measurement of TOC 75
  2.2.3 Automatic Detection of Surface Oil 75

2.3 Automatic Measurement of Water Quality for Process Control of the Secondary Treatment 75

2.4 Automatic Measurement of Water Quality for Process Control of the Tertiary Treatment 76
  2.4.1 Measurement of Nutrients such as Ammonia Nitrogen by Automated Colorimetric Analysis 76
  2.4.2 Automatic Measurement of Ammonia and Nitrate by Electrodes 76
  2.4.3 Use of Turbidimeters 77
  2.4.4 Others 77

2.5 Future Aspect of Study 78

2. AUTOMATIC WATER QUALITY MEASUREMENT FOR WASTEWATER TREATMENT

2.1 INTRODUCTION

In the field of wastewater treatment, measurement of water quality is necessary for a variety of purposes, such as water quality monitoring of industrial wastes discharged into the sewerage system, water quality monitoring of influent to sewage treatment plants, process control of the secondary and tertiary (advanced) treatment, and water quality monitoring of effluent from sewage treatment plants. Virtually all of these measurements are now performed by manual sampling and analysis. In many cases, however, automatic continuous measurement is needed. Water quality parameters for which continuous measurement can be now used are very limited except in the case of comparatively clean treated water. Water temperature, pH, oxydation-reduction potential, dissolved oxygen and sludge density, are parameters which can be measured without much trouble. Even in measuring these parameters, considerable maintenance and inspection are required.

Major points that need a special consideration in the development of automatic continuous measurement of sewage quality are as follows:

(a) Growth of slime

Because sewage normally contains a lot of organic matters and is suitable for propagation of microorganisms, biological slime tends to grow inside the piping or other portions of the equipment contacting sample water. Slime not only causes clogging but interferes measurement.

(b) Effects of suspended solids

When measuring water quality of raw sewage containing high suspended solids, consideration much be given to minimize clogging at sampling portion or inside the piping. The use of automated colorimetric analysis is difficult when the level of suspended solids is high.

(c) Existence of interferences

Generally speaking, the composition of sewage is more complex than those of river water or industrial wastewater and therefore masking of interferences is more critical.

Thus automatic continuous measurement of sewage water quality is more difficult than that of river water, etc. But as its necessity is very high, efforts are made to develop and put into use devices for this purpose.

2.2 AUTOMATIC WATER QUALITY MONITORING OF RAW SEWAGE

2.2.1 DEVELOPMENT OF AUTOMATIC CYANIDE MONITOR

Cyanide is one of the toxic substances which may be discharged into the sewerage system. By controlling its discharge, reduction of discharge of heavy metals from industries such as plating works can be attained. Therefore, the Ministry of Construction gives the first priority to the development of automatic cyanide monitoring device.

The Ministry has been supporting studies on developing automatic free cyanide monitoring device including field tests using model devices during the past two years. In measuring cyanide by selective-ion electrodes, it is the most important to prevent the effects of interferences. Sulphide, iodide, bromide, sulphite, thiosulphite, thiocyanate and organic substances with SH-radical are the major interferences which may be present in sewage. However, in the case of normal sewage, only sulphide will bequire consideration. Fig. 2.1 shows the degree of interferences of anions other than sulphides. Sulphides were removed first from samples by addition of cadmium nitrate. And then, each sample was devided into two parts. One part of the sample was then anion exchanged. The same amounts of cyanide were added to both parts of the sample, and the concentrations of cyanide were measured by an ion electrode. The figure shows that measurement of raw sewage is possible when masking of sulphide is performed. In the case of primary effluent of night soil treatment plants, interferences of anions other than sulphides are perceived. Most of these are considered to be organic substances with SH-radical, such as thioglycollic acid. For the masking of sulphides, cadmium, bismuth, zinc and lead are effective. Above all, the most stable masking is possible by adding cadmium, so cadmium mitrate has been used in field tests. But because adding these heavy metals to sewage even in a small quantity is not desirable, other methods of masking are being studied, such as one using filter paper on which cadmium is coated by vacuum evapolation or one utilizing other masking agants than these heavy metals. In the former method, cadmium can be recovered easily but the amount of sulphur to be removed is limited and is not suitable for sewage containing a large quantity of sulphides. As masking agents other than heavy metals, it has been found that sodium nitroprusside and several types of nonionic and anionic polymers (if added in a high concentration) are effective, but they are also toxic. Thus, studies must be continued to find more proper masking methods of sulphides. For masking of organic substances with SH-radical, it has been proved that sodium molybdate is effective.

Fig. 2.2 shows the block diagram of model device. In the early stage of field tests at the Kisshoin Sewage Treatment Plant, Kyoto, several points to be improved were discovered. Most essential of them was the phenomenon that the surface of the sensing element of the cyanide electrode was deteriarated even if masking of sulphides had been carried put. To cope with this, the electrode was modified as shown in Fig. 2.3 so that mechanical scraping can be performed continuously.

After the above modification, test runs have been continued over one year until today during which maintenance work such as calibration has been done every two weeks. Table 2.1 shows causes by which data could not be obtained during 1974 and the ratios of the periods during which data were lost to the total period of operation. The period during which data could not be obtained is about 37% of the total period. This is not very small percentage but most of the causes can be eliminated by more frequent inspection and maintenance work and it is not very difficult to reduce the percentage to less than 10%.

Though there remain some problems with masking agents, we could develop a usuable equipment for automatic free cyanide measurement. However, as pretreatment standards provide for regulations for total cyanides including complex cyanides in most cases, there is a need for measuring total cyanides, not only for free cyanide. Several devices for automatic measurement of total cyanides are already on the market, but they have not virtually been applied to the measurement of cyanides in sewage. It is considered that a lot of troubles will be encountered in these devices for actual measurement. Therefore, a project is now being carried out to remodel them through field tests.

2.2.2 AUTOMATIC MEASUREMENT OF TOC

The Area River Basin (Left Basin) Sewerage Authority is conducting automatic measurement of TOC to determine organic loadings to their sewege treatment plant. Similar measurement is carried out in Osaka Prefecture for water quality monitoring of storm water overflow from combined sewer. Models used are different but both models suffer from clogging at sampling portions and piping. The former model which sample water is continuously injected by a peristaltic pump, was faced with such problems as fluctuations in measurements caused by non-uniform entrance of SS, fluctuations in the base line, precipitation of minerals on the inlet port of the furnace, and, during earlier stages, corrosion of the piping material due to hydrochloric acid. On the other hand, in the case of Osaka Prefecture where intermittent measurement method is used, though the sample line is cleaned by water while measurement is not carried out, cloggings of the injection valve occurred frequently. In any case, troubles by SS are unavoidable in the automatic measurement of TOC of raw sewage. At this moment there are no alternatives but to reduce these troubles by frequent maintenance and inspection.

2.2.3 AUTOMATIC DETECTION OF SURFACE OIL

Development of automatic devices for surface oil detection for use in sewers and pumping stations is now being continued by studies financially supported the Ministry of Construction. Methods under study are measuring fluorescence emitted by the radiation of ultraviolet rays, measuring reflecting light by the radiation of infrared rays of two wave lengths, and measuring changes of mutual inductance.

2.3 AUTOMATIC MEASUREMENT OF WATER QUALITY FOR PROCESS CONTROL OF THE SECONDARY TREATMENT

Parameters requiring automatic measurement to conduct process control of the secondary treatment include dissolved oxygen, concentration of organic matters expressed by TOC, etc., MLSS, sludge density, etc. Of these parameters, measurement of concentration of organic matters expressed by TOC, etc. has many difficulties. But other items are basically in the stage of practice today, though there remain problems including necessity for frequent maintenance and inspection, and insufficient accuracy of measurement.

There is no sewage treatment plant in Japan in which air to aeration tanks is controlled automatically in accordance with the concentration of dissolved oxygen in tanks. However, studies for them are being performed widely including full scale experiments. It has been affirmed that automatic measurement of dissolved oxygen in aeration tanks is possible if calibration of electrodes and cleaning of electrode surfaces are sufficiently done.

For measurement of sludge density, the method using ultrasonic waves is extensively used. It is said that since the development of devices for eliminating interferences of air bubbles, measurement accuracies have been improved.

2.4 AUTOMATIC MEASUREMENT OF WATER QUALITY FOR PROCESS CONTROL OF THE TERTIARY TREATMENT

2.4.1 MEASUREMENT OF NUTRIENTS SUCH AS AMMONIA NITROGEN BY AUTOMATED COLORIMETRIC ANALYSIS

Water quality measurement of secondary or tertiary effluents can be conducted by means of automated colorimetric analysis, because SS is low and slime growth is less.

In the tertiary treatment pilot plant established in the Toba Sewage Treatment Plant in Kyoto, continuous colorimatric analysis of ammonia nitrogen is carried out, for automatic control of the break-point chlorination process. Major points to which special attention was paid in the development of this device are as follows: (1) output is linear within the range between 0 and 20mg/l of ammonia nitrogen; (2) dual filtering devices are installed so that they can be cleaned alternative; (3) disinfection device is installed to prevent growth of slime; (4) color of sample water is compensated; and (5) automatic calibration is performed at least once a day.

The method used is the salycylate-dichloroisocyanurate reaction with ammonia (1). This method has an advantage that the color development is stable even in high concentration of ammonia nitrogen, as compared with the Nesslerization Method and the Phenate Method. The block diagram of this device is shown in Fig. 2.4. Samples are heated to approximately 85°C for a short period to prevent slime and algae growth. But when measuring treated water from the break-point chlorination process, disinfection by heating is not performed, because measured values become smaller when samples are heated and of slime control is unnecessary. To filter sample water, ceramic filters with a mean pore size of 30μ are used. Backwash by air and water is conducted automatically. Comparison of analytical values by this device with those by the manual Nesslerization Method is shown in Fig. 2.5. According to the Figure, when the concentration of ammonia nitrogen becomes about 20mg/l, analytical values by automatic measurement tend to be smaller. This is because calibration is performed at 0mg/l and 10mg/l, though absorbance becomes somewhat non-linear when concentration exceeds about 15mg/l. But as a whole the results are satisfactory and automatic control of the treatment process is conducted satisfactorily by means of feed forward control using this device.

For automatic control of coagulant dose in chemical clarification for phosphorus removal, an automatic measurement device of hydrolyzable phosphorus based on similar principle is now being developed. For this device disinfection by ultraviolet rays to prevent slime growth is also considered. Concerning filtration, synthetic resin filters are examined, not only ceramic ones.

2.4.2 AUTOMATIC MEASUREMENT OF AMMONIA AND NITRATE BY ELECTRODES

In measuring ammonia and nitrate, use of ion electrodes is one of the best methods. Therefore, in the above-mentioned pilot plant for the break-point chlorination process, surveys on application of these measuring devices are conducted. Since nitrate ion electrodes are interfered by chlorides and others, they are difficult to be used for measuring nitrate in the effluent from break-point chlorination process. Therefore, they are now used only for measuring nitrate in the influent.

Measurement of ammonia is carried out for both influent and effluent. In the latter, because there exists a possibility that the membrane is damaged by residual chlorine and chloramines, measurement is done after passing samples through activated carbon columns. The results of experiments so far shows that the reliability of the measurement by ammonia electrode is somewhat lower compared with that by colorimetric analysis. Another disadvantage is the short life of the electrode. It lasts only a few months when used continuously.

Experiments of measurement of nitrate in the secondary treatment water by means of nitrate electrodes have just begun. The results obtained so far are not so encouraging. One of the reasons for this is insufficient maintenance and inspection, so experiments are being continued with more frequent maintenance and inspection. If continuous measurement of nitrate in the secondary effluent becomes possible by means of ion electrodes, it will be utilized for automatic control of biological denitrification process.

2.4.3 USE OF TURBIDIMETERS

The Sewage Works Bureau of Tokyo Metropolitan Government is conducting surveys at its pilot plant for granular media filtration of the secondary effluent which is built in the Morigasaki Sewage Treatment Plant, in order to use turbidimeters for automatic control of granular media filtration. In this pilot plant various types of turbidimeters are installed. At present, a falling stream type turbidimeter (scattering light of 23°) is being used for measurement of influent and effluent turbidity, while for measuring turbidity of backwash waste water the surface scattering type one is introduced. At though there are some problems of slime growth in dealing with less treated water, there are virtually no trouble in measurement. There are corelations between turbidity and SS as shown in Fig. 2.6 depending on the type of the water measured.

In the pilot plant for the tertiary treatment in the Toba Sewerage Treatment Plant in Kyoto, the surface scattering type turbidimeters are now used for measuring turbidity of influent to filters, effluent from filters and effluent from chemical clarifier. Also, studies are being conducted concerning control of fitter backwash by using turbidity of backwash waste water.

2.4.4 OTHERS

In the pilot plant for filtration in the Morigasaki Sewage Treatment Plant, Tokyo, in addition to the abovementioned measurement of turbidity, automatic measurement of TOC, TOD, dissolved oxygen and residual chlorine is performed. Measurement of residual chlorine is conducted when chlorine is dosed to control slime growth.

TOC and TOD are of types in which sample water is continuously injected by a peristaltic pump. They need remodeling including increase in diameter of pipes at inlet port to the furnace, increase in gas cooling capacity, and providing of activated carbon column to adsorb hydrochloric gas.

In measuring dissolved oxygen, after flow cells have been introduced which allow higher contact flow velocity, the maintenance of about once a year has been sufficient.

2.5 FUTURE ASPECT OF STUDY

Automatic measurement of water quality for process control of the tertiary treatment gives few problems in the development of devices for it, because it deals with comparatively clean water. Thus a various kinds of devices have already been developed and more will be devised in the future when necessary. On the other hand, automatic measurement of water quality of row sewage and industrial wastewater discharged into the sewerage system has seldom been conducted in spite of its high necessity, and there are almost no devices available for it. Monitoring of heavy metals and other toxic materials is one of the most important factors for the operation and maintenance of a sewage treatment plant. Therefore, studies in this field should be positively promoted in the future. Such studies should include, in addition to those on measuring methods themselves, improvement in methods of sampling and filtration, methods of slime control, and methods for dissolving heavy metals contained in solids.

REFERENCE

1) Searcy, R.L., Reardon, J.E. and J.A. Foreman, "A New Photmetric Method for Serum Urea Nitrogen Determination", American Jour. of Medical Technology, Vol. 33, No. 1, 1967.

Table 2.1 Inventory of Data Loss During 6 Months Operation of Automatic Cyanide Monitor

Causes of data loss	Period of data loss (hrs)	Percentage of data loss (%)
Malfunctions of recorder chart drive	92	2.1
Running out of chart paper	156	3.6
Running out of recorder ink (pH)	176	4.1
Running out of recorder ink (CN-pH)	429	9.9
Power failure (main source)	96	2.2
Fuse blow	151	3.5
Breakage of pH electrode	138	3.2
Clogging of strainer	38	0.9
Malfunction of vacuum pump	42	1.0
Running out of NaOH solution	80	1.9
Shortage of inner solution of reference electrode	196	4.5
Total	1,594	36.9
Total period of operation	4,320 (hrs)	

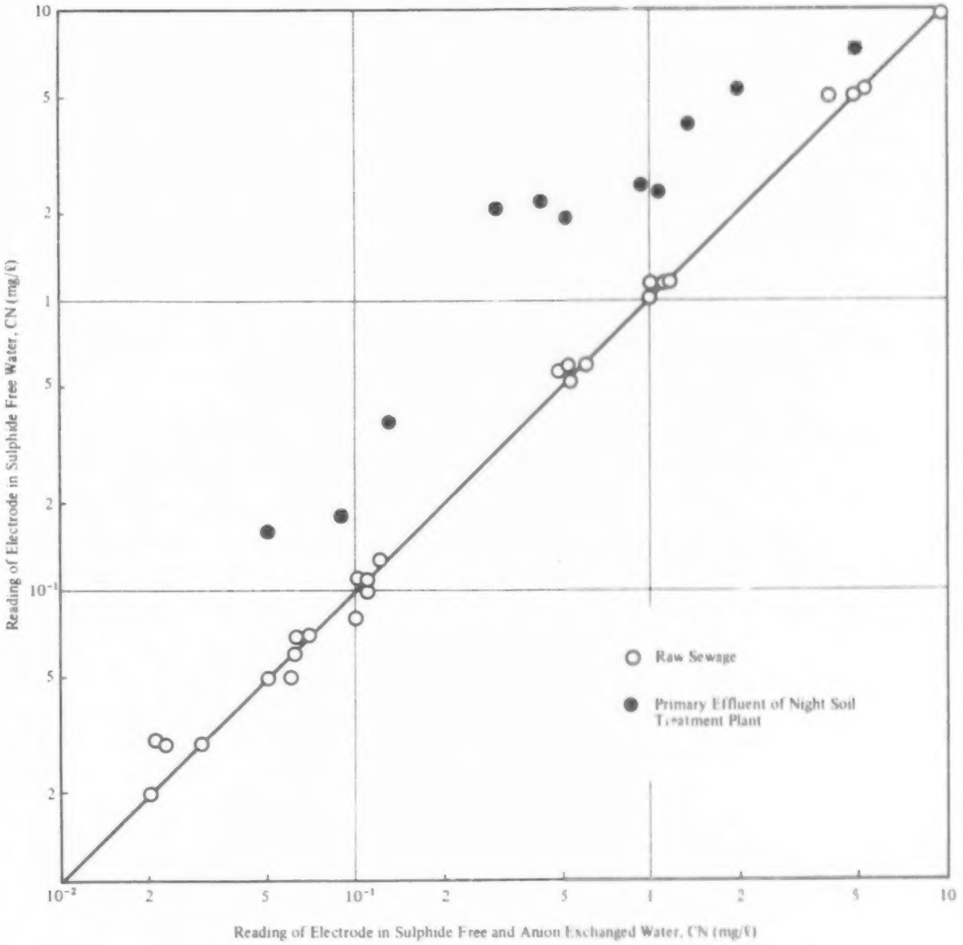


Fig. 2.1 Interferences of Anions Other Than Sulphides with Cyanide Measurement by Ion-Selective Electrode

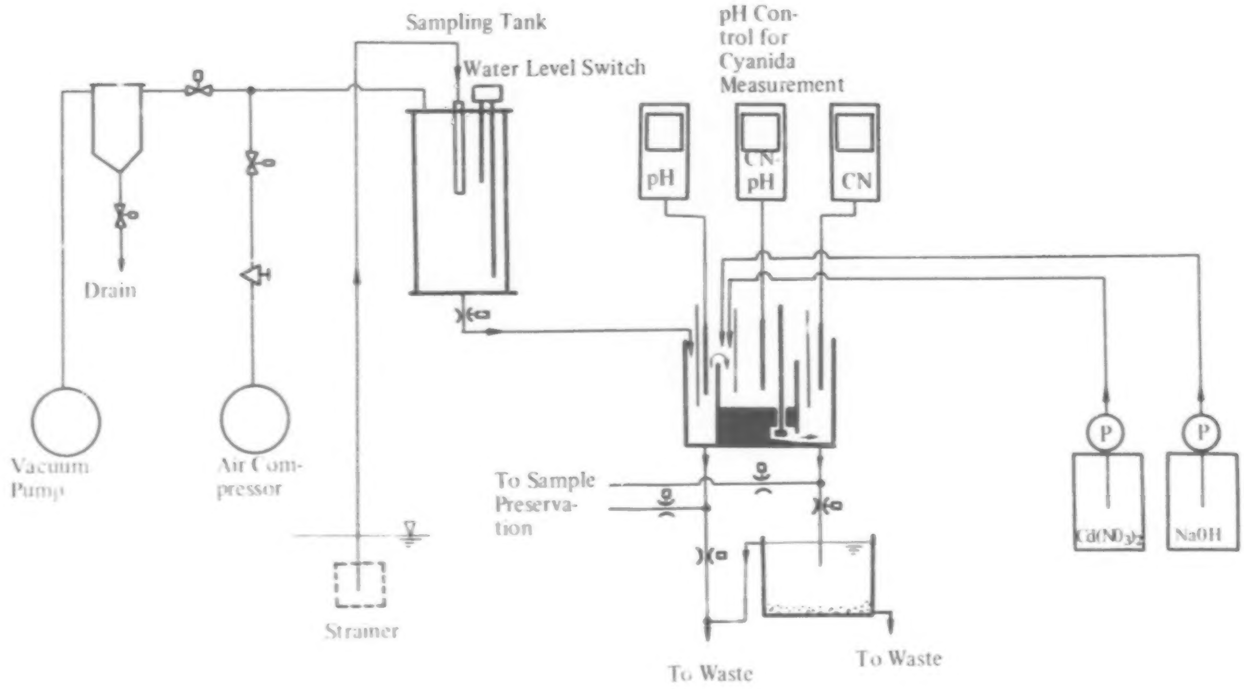

Sampling Tank
Water Level Switch
pH Control for Cyanida Measurement
pH
CN-pH
CN
Drain
Vacuum Pump
Air Compressor
Strainer
To Sample Preservation
To Waste
To Waste
P
P
Cd(NO3)2
NaOH

Fig. 2.2 Block Diagram of Automatic Free Cyanide and pH Monitor

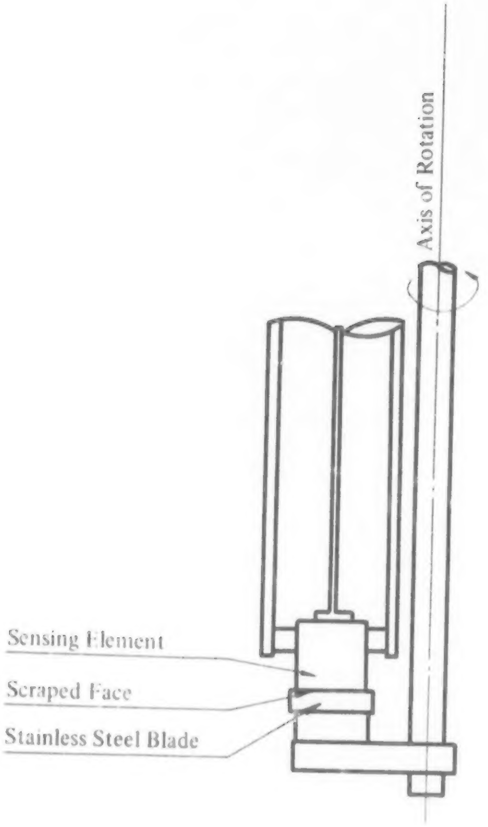

Axis of Rotation
Sensing Element
Scraped Face
Stainless Steel Blade

Fig. 2.3 Cyanide Elecfrode with Continuous Scraping Device

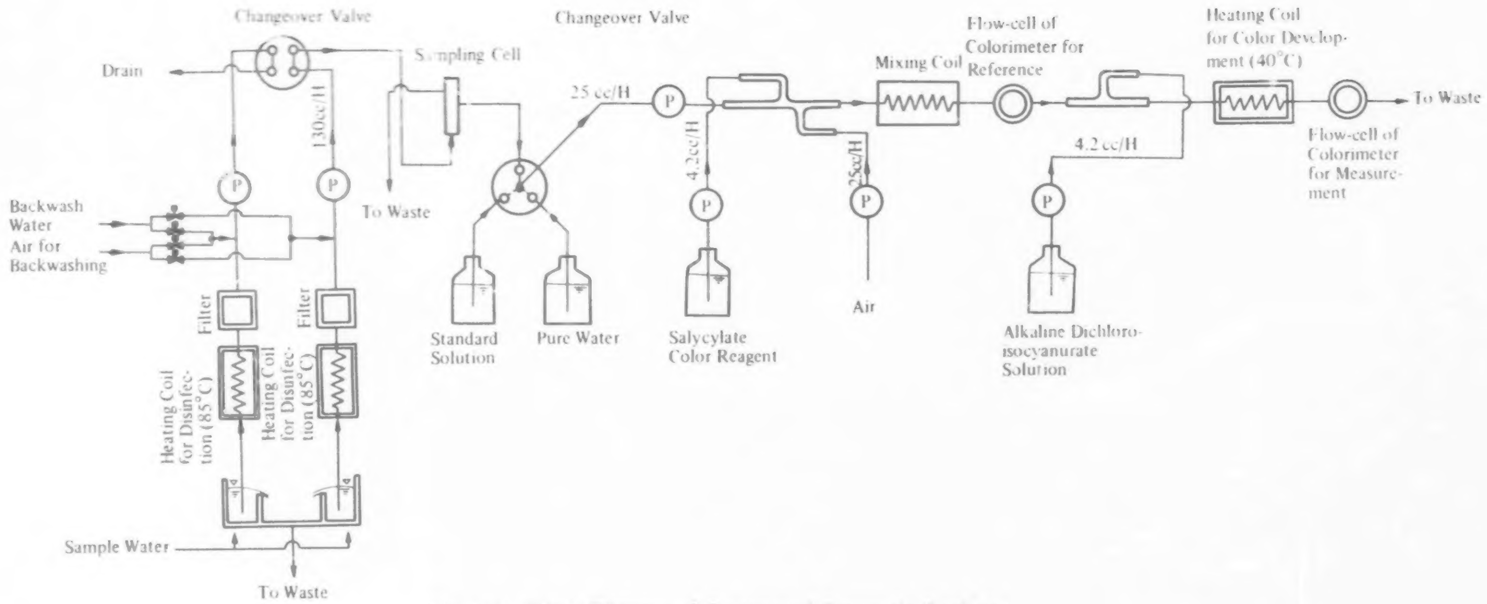

Changeover Valve
Drain
130cc/H
Sampling Cell
To Waste
Changeover Valve
25 cc/H
P
4.2cc/H
25cc/H
Mixing Coil
Flow-cell of Colorimeter for Reference
4.2 cc/H
Heating Coil for Color Development (40°C)
To Waste
Flow-cell of Colorimeter for Measurement
Backwash Water
Air for Backwashing
Filter
Filter
Heating Coil for Disinfection (85°C)
Heating Coil for Disinfection (85°C)
Standard Solution
Pure Water
Salycylate Color Reagent
Air
Alkaline Dichloroisocyanurate Solution
Sample Water
To Waste

Fig. 2.4 **Block Diagram of Automated Ammonia Analyzer**

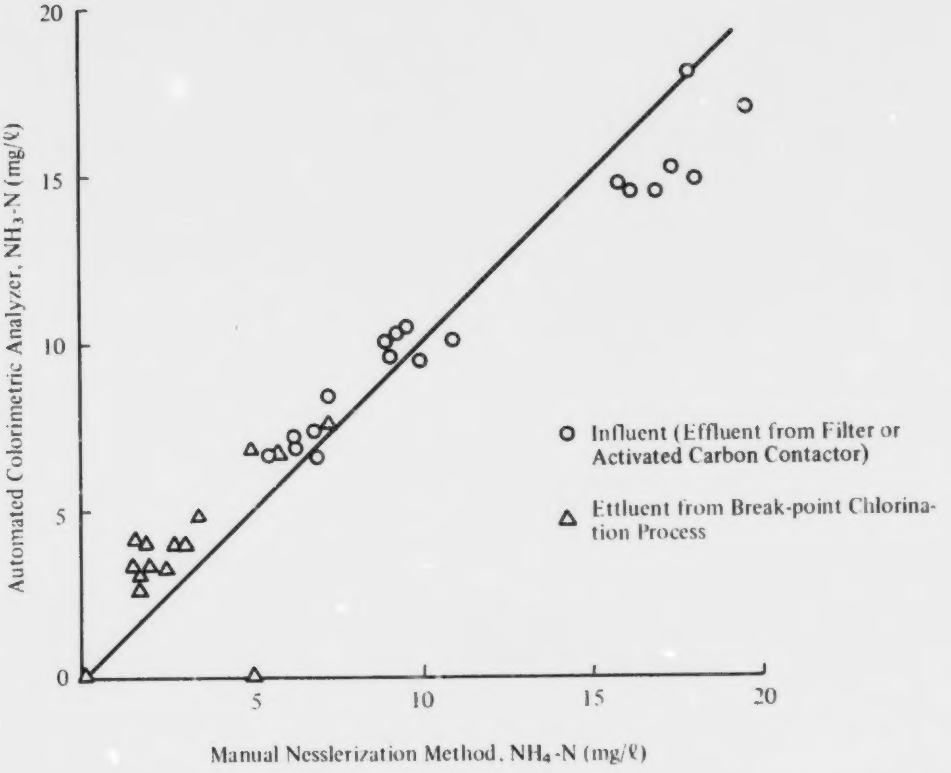


Fig. 2.5 Comparison of Ammonia Nitrogen Concentrations obtained by Manual and Automated Analyses

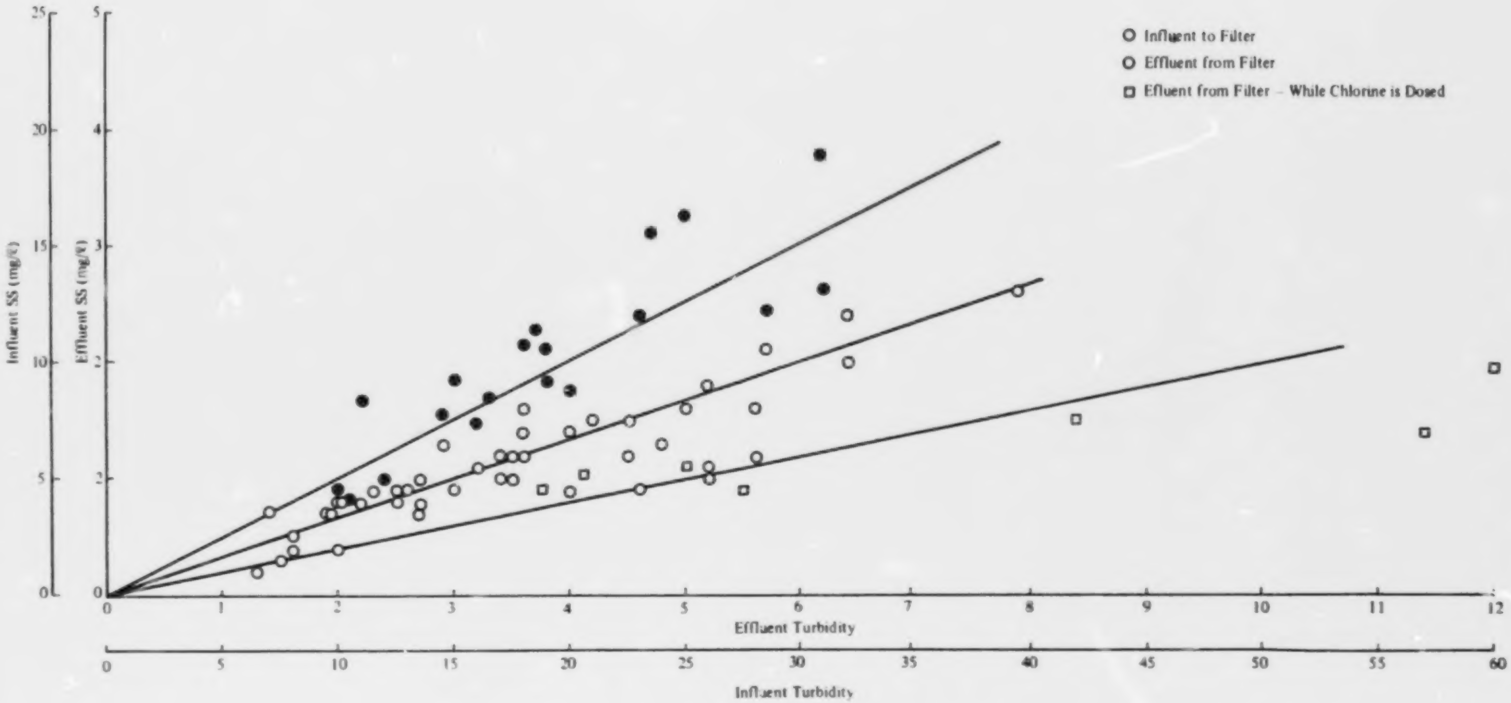


Fig. 2.6 Relationship between Turbidity Measured by Falling-Stream Type Turbidimeter and Suspend Solids

Fourth US/JAPAN Conference
on
Sewage Treatment Technology
Paper No. 4

RECENT PROGRESS IN ENVIRONMENT QUALITY IN JAPAN

October 29, 1975
Washington, D. C.

Ministry of Construction
Japanese Government

CONTENTS

I NATIONWIDE SURVEY ON MERCURY AND PCB POLLUTION 89
1.1 Findings About Mercury Pollution 89
1.1.1 Fish and Shellfish 89
1.1.2 Quality of Water 90
1.1.3 Bottom Sediments 91
1.1.4 Soil and Farm Products 91
1.1.5 Controls at Sources of Pollution 92
1.2 Findings about PCB Pollution 93
1.2.1 Fish and Shellfish 93
1.2.2 Quality of Water 94
1.2.3 Bottom Sediments 94
1.2.4 Soil and Farm Products 95
1.2.5 Controls at Sources of Pollution 95
1.3 Overall Evaluation 96
II CASE STUDIES ON MERCURY AND PCB POLLUTION 97
2.1 Case Studies on Mercury Pollution 97
2.1.1 The Case in Tokuyama Bay 97
2.1.2 The Case in Jinzu River 103
2.2 Case Studies on PCB Pollution 103
2.2.1 The Case in Lake Biwa ....
2.2.2 The Case in Tsuruga Bay ....
III REVISIONS OF ENVIRONMENTAL AND EFFLUENT STANDARDS ON MERCURY AND SETTINGS OF ENVIRONMENTAL AND EFFLUENT STANDARDS ON PCB 105
3.1 Mercury 105
3.1.1 Reasons for Amendment 105
3.1.2 Main Points of Revision 107
3.2 PCB 108
3.2.1 Basic Principles 109
3.2.2 Consideration of Relevant Factors 109
3.2.3 Environmental Quality Limit on PCBs 110
3.2.4 Waste Water Discharge Standards 111

3.2.5 Provisional Standard for Removal of PCB-Contaminated Sediments 112
IV PROVISIONAL STANDARDS FOR REMOVAL OF CONTAMINATED SEDIMENTS 112
4.1 Mercury-contaminated Sediments 112
4.1.1 For Sea Area 112
4.1.2 For Rivers and Lakes 113
4.2 PCB-contaminated Sediments 113

I NATIONWIDE SURVEY ON MERCURY AND PCB POLLUTION

A nationwide survey in Japan on mercury and PCBs (polychrolinated biphenyls) pollution was done during the period from July '73 to March '74.

Undertaken at a time when contamination of fish, shellfish and the environment by mercury and PCBs is becoming worse, the survey was aimed at obtaining factual data on the actual state of affairs and investigating the causes of pollution.

The ultimate objectives were, of course, to assure the safety of fish and shellfish and establish the basic countermeasures required to clear the environment of mercury and PCBs.

Participating in the survey were the Environment Agency, the Ministry of International Trade and Industry, the Ministry of Transport, and the Ministry of Construction. Prefectural governments across the country cooperated in the survey. The gist of the findings and an overall evaluation are given below:

1.1 FINDINGS ABOUT MERCURY POLLUTION

1.1.1 Fish and Shellfish

Tested for mercury were 22,403 samples of 303 species, of fish and 616 samples of plankton which were collected from 124 inland bodies of water and 144 sea zones.

The majority of the species covered were cleared for fishing and marketing, as their mercury levels were found to be below the provisional standards. But some fish species caught in Minamata Bay and Tokuyama Bay, where voluntary fishing restrictions are already in force, exceeded the provisional limits. So did some species caught in the sea off Naoetsu in Niigata Prefecture — such as "ishimochi (Argyrosomus argentatus)","kanagashira (Lepidotrigla microptera)", "magarei (Limanda yokohamae)" and "akamutsu (Upeneus bensasi)" — and some species caught in the inner parts of Kagoshima Bay — such as "tachiuo", "madai", "akana", "anago" and "maaji". Voluntary fishing curbs have been put into effect on these species following the earlier discovery of high mercury presence by Niigata and Kagoshima prefectural governments.

Fish and shellfish in the sea off Naoetsu and inner parts of Kagoshima Bay must be kept under continued surveilance. The fact that investigations of bottom sediments failed to reveal the causes of pollution in these areas calls for a more intensive probe into the mechanism of contamination, so that necessary countermeasures can be formulated.

Fish caught in nine rivers showed mercury levels well above the provisional standards. The earlier surveys of these rivers had resulted in similar findings except for one in Nagasaki Prefecture, and the inhabitants living along the rivers have already been given dietary guidance by the authorities.

The contamination of these rivers may be spontaneous since many of them flow through regions where there has historically been some pollution originating from mercury mines. A close inquiry, nevertheless, is required to determine the precise cause of river pollution and the effectiveness of possible countermeasures by implementing such steps as reducing the discharge of pollutants at known sources, removing bottom sediments that are dredged where necessary, plus conducting regular checks on fish and shellfish in the interim.

Surveilance of foodstuffs is also being maintained in fish markets to assure the safety of fish and shellfish that are channeled to consumers.

1.1.2 Quality of Water

The water aspect of the mercury survey involved 3,768 samples taken from 331 rivers, 156 ports and harbours, and 147 sea areas. Seventy-six samples, or two percent of the total, were found to contain total mercury levels exceeding 0.005 ppm.

The samples topping the environmental quality standards were from the Mukagawa River in Hokkaido, the upper reaches of of the Tone River near Shibukawa in Gumma Prefecture, the Shingashi River in Tokyo, the Torigai River in Osaka (a tributary of the Kanzaki River), Minamata Bay, and a canal linked with Toyama Port. The causes of pollution in these areas need to be clarified so that the necessary countermeasures can be instituted.

1.1.3 Bottom Sediments

A total of 5,186 samples of bottom sediments were examined for mercury. They were collected from almost the same areas where the water-test samples were taken — 332 rivers,155 ports and harbours, and 148 sea zones. Samples from 16 rivers and nine ports and harbours — i.e., 120 samples, or 2.3 percent of the total — registered mercury values in excess of the provisional standards for removal of polluted sediments.

Dredging work has already been completed or is in progress in most of these bodies of water. Sections that need dredging should be designated promptly in other areas after detailed investigations.

1.1.4 Soil and Farm Products

Samples of soil were tested for mercury at 707 spots --ordinary tests at 469 places and close examinations at 58 places in the vicinity of factories handling mercury.

Values recorded in ordinary tests, in terms of total mercury, ranged from ND (not detectable) to a maximum of 5.36 ppm, the great majority of them being below 1.5 ppm. It seems safe to conclude from this that densities of total mercury contained in common specimen of soil is less than 1.5 ppm.

The samples were analysed for organic mercury. But no appreciablelevels of methyl mercury and ethyl mercury were detected.

The test produced readings ranging from a low of 0.07 ppm to a high of 67 ppm, with the great majority of the samples being below 1.5 ppm. Practically no methyl mercury nor ethyl mercury was detected.

Tests on farm products were conducted at 702 spots —ordinary tests at 464 places and scrutiny at 238 places.

Samples of unhulled rice involved in ordinary tests showed values ranging from ND to a high of 0.17 ppm. The great majority of the recorded readings were below 0.01 ppm. Methyl mercury and ethyl mercury were practically nonexistent.

Total mercury readings resulting from close examinations ranged from ND to 0.12 ppm, the great majority of them below 0.01 ppm — not necessarily high compared with the results of ordinary tests. Practically no methyl or ethyl mercury was in evidence.

1.1.5 Controls at Sources of Pollution

Appropriate treatment or disposal of waste water and refuse is imperative to deter further environmental disruptions.

The situation has remarkably improved with regard to waste water — with strict controls being enforced across the country — instead of in limited bodies of water as before, under the discharge standards established in May, 1971, on the basis of the Water Pollution Control Law.

The discharge standards were strengthened further in August 1974. Especially noteworthy is a fact that the main sources of mercury pollution — factories that turn out caustic soda by using mercury and vinyl monomer and acetoaldehyde by using acetylene — have been shut down or have carried radical changes— either in production processes or by installing self-containing waste-water facilities.

Factories that were built after the establishing of custody and disposal standards on industrial refuse in September, 1971, in line with the Waste Disposal and Public Cleaning Law are being given guidance so that their refuse is properly disposed of in accordance with these rules.

As for factories that existed before the standards were fixed, the assumption is that as a general rule, their refuse is being safely gotten rid of or adequate countermeasures are in effect. There are cases in which refuse is buried or in custody in the factory grounds, some of it outside the grounds. There seems to be no problem, judging from the quality of water in the neighbouring body of water. There are also factories where concrete, clay and other materials are as covers to keep pollutants from flowing or seeping away when it rains. But available information is, admittedly, still insufficient in some cases.

1.2 FINDINGS ABOUT PCB POLLUTION

1.2.1 Fish and Shellfish

Checks on fish and shellfish were carried out in 20 bodies of water — all of which had shown questionably high levels of PCBs in the previous surveys — with 3,369 samples being tested. The estuary of the Tama River in Tokyo and the Nagara River in Aichi Prefecture, in addition to the nine areas where self-restrictions on fishing were instituted in 1973, were found to be more polluted than allowed under the provisional standards. Similar curbs have been enforced in the heavily contaminated Tama and Nagara river sections following earlier findings by the responsible prefectural governments.

Factories handling PCBs have existed along these rivers in the past, and their pollution is fully accountable from the volumes of PCBs consumed in the plants and the extent of contamination of bottom sediments. What needs to be done now, therefore, is to make sure that stocks of PCBs are recalled and kept in the hands of the authorities until they are disposed of. Needless to say, the ban on their use, to be detailed later, is imperative.

Of particular importance is the installation of sophisticated equipment for material screening and waste-water disposal in used-paper recycling plants since water discharged by these plants sometimes contain minute amounts of PCBs originating from pressure-sensitive copying paper and other types of secondhand paper. Removal of contaminated bottom deposits in accordance with relevant standards, to be fixed shortly, and other environmental cleaning measures are promptly for, with detailed surveys made mandatory when necessary.

Appropriate levels of surveilance are also maintained at fish markets to ensure the safety of fish and shellfish sold there.

1.2.2 Quality of Water

Assayed for PCBs as affecting the quality of water were 1,281 samples from 208 rivers, 28 ports and harbours, and 46 sea areas where fish and shellfish contamination had posed a problem in previous surveys or where quality of water examinations had produced 1 ppm or more. Fifty-five samples, 4.3 percent of the total, logged levels higher than 0.0005 ppm. These were collected from 18 rivers and four sea zones. Inquiries into the causes of pollution in these cases are in order, with necessary countermeasures to be taken afterward.

Compared with the previous year's findings, the situation improved on the whole, with reductions recorded both in terms of the number of contaminated samples and PCB densities.

1.2.3 Bottom Sediments

Bottom sediment tests were held on 1,789 samples from 258 rivers, 38 ports and harbours, and 58 sea areas. Samples from 14 bodies of water showed maximum PCB readings in excess of 50 ppm. Maximum levels were between 10 and 50 ppm in 37 other areas and between five and 10ppm in another 21.

Sediment removal work has been completed in some of these regions. As for the remainder, removal or other countermeasures are called for on the basis of the provisional removal standards to established shortly, with detailed investigations to be carried out where necessary.

Exceedingly high levels of PCB pollution were recorded in the previous year's survey, topping 10,000 ppm in some places, but countermeasures have been taken in the areas concerned. As a result, the latest survey did not such extraordinary peaks.

1.2.4 Soil and Farm Products

PCB checks on soil were conducted on samples from 105 places in the vicinity of factories and other establishments handling PCBs in five prefectures and two major cities with a greater measure of self-autonomy accorded by law. Recorded densities ranged from ND to 59 ppm, the great majority of them below 0.1 ppm.

Farm products were found to be practically free of PCBs.

1.2.5 Controls at Sources of Pollution

A total ban has been imposed on the use of PCBs in manufacturing such goods as copying paper, paint and ink. The use of PCBs in such products as heat exchangers, heaters, transformers and condensors is also prohibited as a general rule. Owners of these equipment already in use are urged to keep them under strict control, and in the case of products in which PCBs serve as a heat medium,manufacturers are under orders to replace them with substitutes.

Directives have been issued to institute proper procedures for the recall and custody of copying paper containing PCBs and make sure that they are not used for reclaimed paper production.

Components containing PCBs are being removed from discarded home electrical appliances with the cooperation of local governments. The Appliance PCB Disposal Association has been set up to collect such components and keep them in custody until they are disposed of, pending the establishment of relevant technology. The association plans to extend its services to the disposal of transformers, condensors and the like disjoined from heavy-duty equipment.

While PCB contamination of waste water has improved on the whole from the previous year as a result of guidance based on the provisional standards, controls will further strengthened under the full-fledged standards to be set shortly.

1.3 OVERALL EVALUATION

As a result of the latest survey, without an equal scale-wise in foreign countries, environmental realities in Japan as regards fish and shellfish, farm products, the quality of water, and soil have been brought to light, practically wiping away anxiety about unknown dangers of mercury and PCB pollution.

With voluntary fishing restrictions introduced in the bodies of water which were found to be heavily polluted, it can be said that a system to avert adverse effects on the health of people through the intake of contaminated fish and shellfish has been set up.

However, as is clear from the findings of the survey, environmental health has yet to be restored in these areas, although there has been improvement at the sources of pollution in the overall picture.

Our target is an environment which would enable people to eat fish and shellfish without any fear of health damage. For this purpose, the relationship between polluting factors and contaminated fish and shellfish must be cleared up.

Polluting elements are traceable through periodical checks, except for some bodies of water where detailed investigations are needed. For the time being, therefore, emphasis should be placed on prompt implementation of measures to improve the quality of water, bottom sediments and reduce the release of pollutants at their sources, as it was pointed out before. Parallel with the effort, necessary detailed investigations should be promoted, with countermeasures based on their findings to be worked out quickly.

II CASE STUDIES ON MERCURY AND PCB POLLUTION

2.1 CASE STUDIES ON MERCURY POLLUTION

2.1.1 The Case in Tokuyama Bay

Tokuyama Bay is a good natural harbour and is located at the western part of Honshu-island, and fronts on Seto Inland Sea. Tokuyama City (population 105,000) and Shin-nanyo City (population 34,000), both are industrial cities, are face at this bay.

Here, Tokuyama Soda Company (1952) and Toyo Soda Company (1956) had been produced caustic soda and chlorine by mercury caustic process.

The total amount of mercury used and the estimated amount which had been discharged into the bay by these two factories are shown in Table 1 :

Table 1

Names of the Co.	Tokuyama Soda Co.	Toyo Soda Co.	Total
Names of the factories	Tokuyama	Nanyo	
Date of the investigation	1973	June 7, 1973	
Total amount of mercury used(t)	703.2	558.6	1,261.8
Mercury discharged into waste water (t)	3.69	2.95	6.64

Since it was already known that the fish and shellfish in this bay were highly contaminated by mercury, from 1.0 to 1.5 ppm, a close survey was undertaken on water, bottom sediment and marine products, when the nationwide survey was done in 1973 as one of the water areas that seriously polluted by mercury.

As to the result of water quality survey,all the 69 samples were "not detectable".

On the contrary, allthe samples of bottom sediments showed mercury existence. (Table 2)

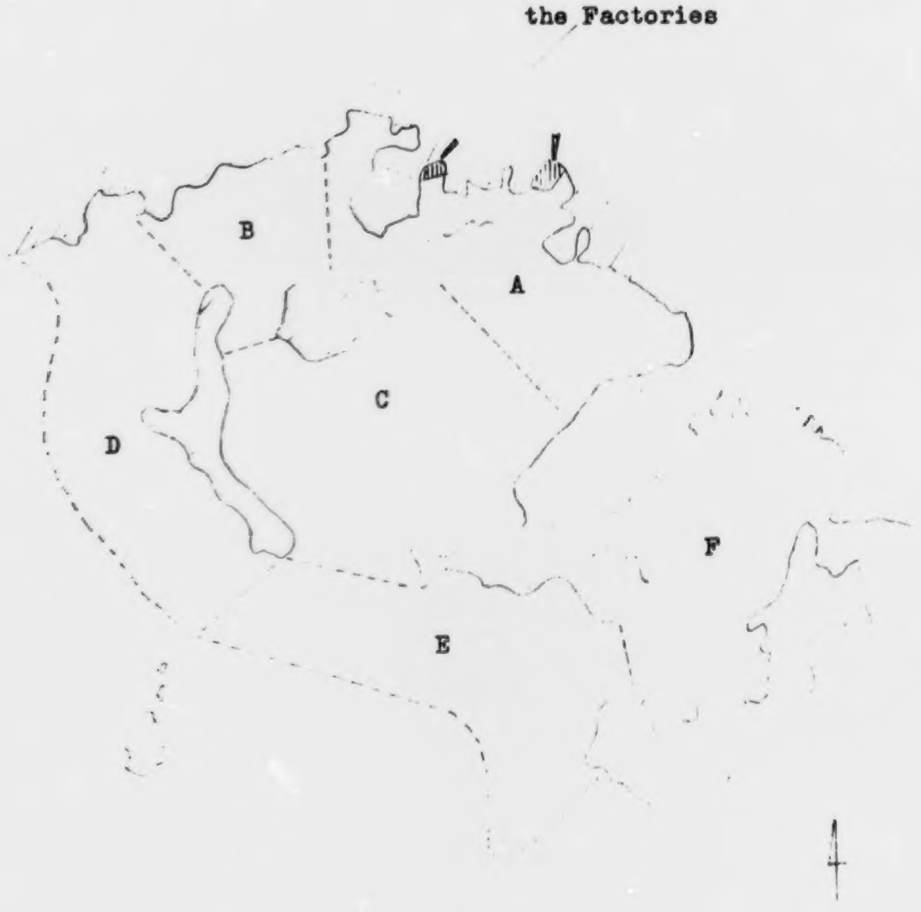
Table 2 The result of bottom sediments of Tokuyama Bay

Lots	Numbers of survey points	Nrs of samples	Total mercury		
			Nrs detec'd	Mean	Min. - Max.
A Sea area	131	131	131	6.64	0.07 - 31.57
	80	80	80	4.56	0.25 - 18.98
B	44	44	44	3.85	0.45 - 7.25
C	121	121	121	2.63	0.13 - 19.50
D	40	40	40	0.91	0.04 - 2.96
Sub-total	416	416	416		
Toyo Soda	2	2	2	5.15	4.16 - 6.14
Tokuyama Soda	2	2	2	13.32	5.83 - 20.8
Tokuyama Soda	1	1	1	0.56	0.56
Sub-total	5	5	5	7.50	0.56 - 20.8
Total	421	421	421		

The Department of Technology of Yamaguchi University estimates according to the results of these survey and also the results of survey boring which were undertaken at the same time that the amount of mercury discharged from these two factories into this bay should be 13 - 14 tons (4 centimetre beneath from the surface of the sea bottom) upto 36 tons (two metre ditto).

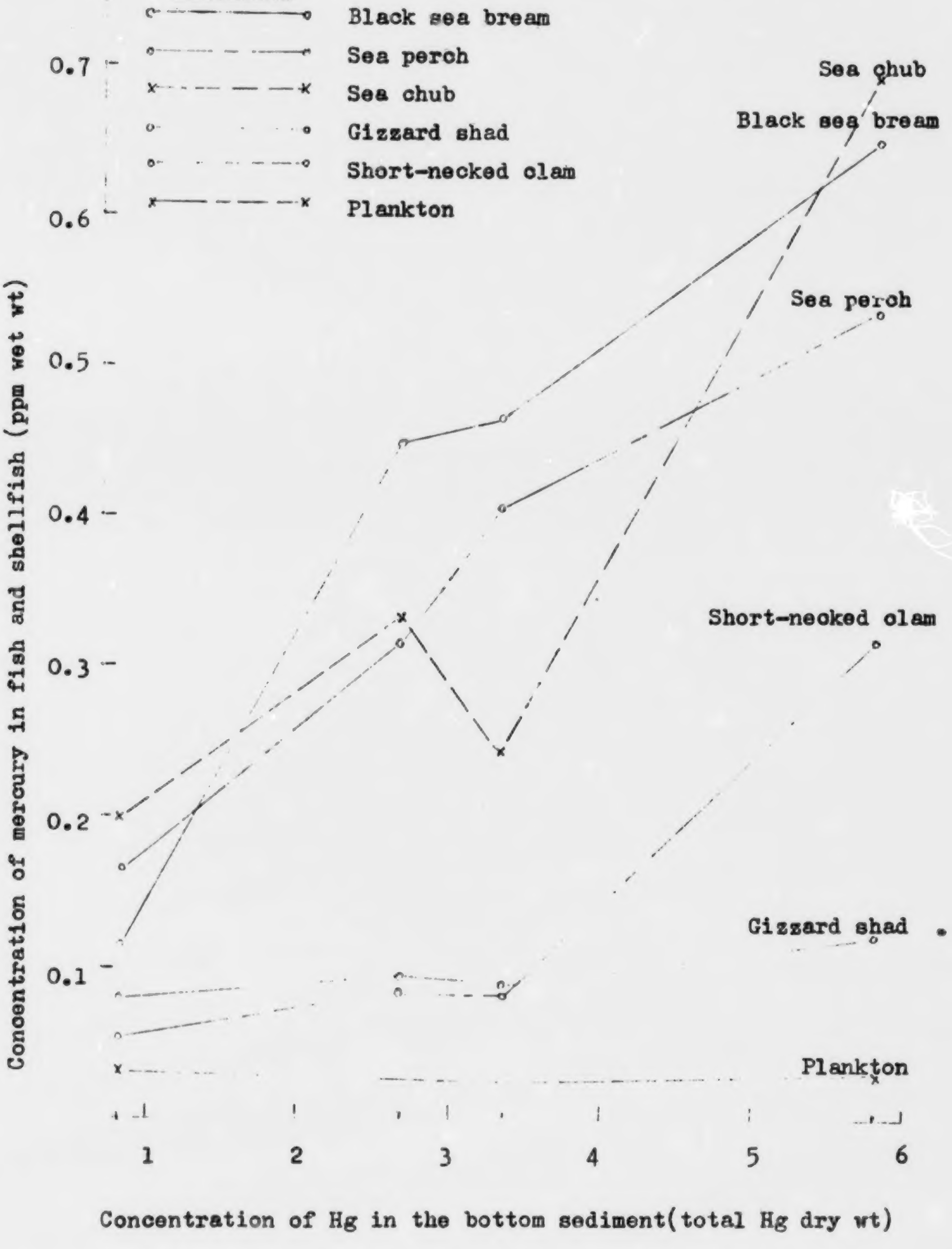
Many fish and shellfish were caught in this bay and analysed as to mercury. Figure 2 shows the relationship between concentration of mercury in fish and shellfish and concentration of mercury of bottom sediments.

Fig. 1 Map of Tokuyama Bay



the Factories
A
B
C
D
E
F

Fig. 2 Relationship between concentration of mercury in fish and shellfish and concentration of mercury of bottom sediments



Also a survey for mercury content in the hair of the inhabitants(60 men and women) living along the coast of Tokuyama Bay. Table 3 shows mercury content in women's long hairs.

Table 3

Sample No	Length of hair(cm)	Conc.(ppm)
No 2 (70 yrs of age)	0 - 10	7.8
	10 - 15	8.0
	15 - 19	8.8
	19 - 22	9.1
	22 - 25	9.1
	25 - 27.5	9.6
	27.5- 30	9.5
	Mean	8.84
No 5 (57 yrs of age)	0 - 10	10.9
	10 - 15	8.1
	15 - 20	7.1
	20 - 25	7.3
	25 - 29	6.9
	29 - 33	6.9
	33 - 37	8.2
	37 - 41	10.7
	41 - 44	13.4
	44 - 47	13.4
	47 - 50	12.9
	Mean	9.62
No 15 (23 yrs of age)	0 - 4	4.61
	4 - 6.5	4.61
	6.5- 9	4.71
	9 - 12.5	4.90
	Mean	4.71
No 16 (52 yrs of age)	0 - 25	5.30
	25 - 35	4.90
	35 - 45	3.73
	45 - 50	3.24
	50 - 55	2.75
	55 - 60	2.35
	Mean	3.71
No 20 (25 yrs of age)	0 - 10	2.35
	10 - 17	2.94
	17 - 24	3.82
	24 - 32	3.94
	Mean	3.26
No 29 (21 yrs of age)	0 - 10	7.36
	10 - 18	8.04
	18 - 24	6.67
	24 - 30	7.36
	Mean	7.36
No 60 (42 yrs of age)	0 - 26	3.14
	26 - 36	3.14
	36 - 46	2.65
	46 - 54	2.94
	54 - 60	2.55
	60 - 65	2.75
	65 - 69	3.24
	69 - 72.5	3.53
	72.5- 76	3.82
	Mean	3,08
No 61 (42 yrs of age)	0 - 10	2.8
	10 - 15	2.8
	15 - 19	2.2
	19 - 23	2.4
	23 - 28	2.6
	28 - 33	2.4
	Mean	2.53

Conclusion

1) Dicharged quantities of mercury from soda-producing factories should indicate the quantities of environmental pollution, but the facts are not so simple because of difficulty to get

continuous records of effluent concetration, and because of the differences of water areal extent or levels of maintenance of factories.

2) There are little chances to detect mercury in water quality survey, and there is a problem as to the quantitative analysis limit in the case of sea water.

3) Mercury contended in bottom sediments could show the quantities of mercury discharged or the state of the waste water treatment of the factory concerned.

4) When mercury concentration in surface potion of bottom sediments doesn't exceed 2 - 2.5 ppm (dry wt), the concentration of mercury in fish meat will not exceed 0.4 - 0.5 ppm (in the case of sea water).

5) Fish are good index for monitoring to pollution because of their bioconcentration of methyl mercury.

6) Generally speaking, much-eating and bottom-inhabited fish indicate greater magnification. Crustacea and plankton don't indicate greater magnification.

7) Hairs of inhabitants along the coast concerned, when analysed, tell the history in the past of mercury pollution for fish and water area concerned.

8) It is scheduled that the bottom sediment of Tokuyama Bay will begin to be removed within this year(concentration over 15 ppm). It is anticipated that we could get the answer whether vastly spreading mercury in the bottom sediments affect fish pollution or high concentration of mercury in limited spots affect fish pollution.

9) Middle-sized fish have 4-5 years of life, and also biological half-period of methyl mercury is relatively long for 1 - 2 years, we should consider that there are time-lags between the concentration of mercury in fish and the environmental clean-up performances.

2.1.2 The Case in Jinzu River

1) Around in the time 1967, it was disclosed that dace in Jinzu River (flows through Toyama prefecture) were highly polluted by mercury(over 3 ppm), and mercury in dace was in the form of methyl mercury.

2) According to the results of surveys, the cause of pollution was suspected to be the pollution caused by a pharmaceutical company situated along the coast of a branch river of Jinzu River.

3) The production of "thimerosal (an organic mercuric disinfectant, sodium ethyl-mercury thiosalicylate)" was rapidly increased in 1968 to 1969.

4) According to the result of the survey held by the Institute of Hygiene of Toyama Prefecture it is disclosed that there were high concentration of mercury(2,300 ppm, including 32 ppm of methyl-mercury)in the bottom sediments near the discharge point of the factory.

5) Halting the production of "thimerosal,merthiolate" and removal of bottom sediment which containing high concentration of mercury, mercury concentration in "ayu (sweetfish)" restored cleanliness after one year, but in the case of dace it took four years to lessen the value of mercury concentration.

2.2 CASE STUDIES ON PCB POLLUTION

Production of PCBs in Japan began in 1954, and by 1970 output reached 11,000 tons per year. But the advent of concern over possible pollution brought the output down to 6,800 tons in 1971, and in 1972 production was totally halted. PCBs had also been imported prior to World War II, although statistics on the quantity are not available. During the period from 1954 to 1972, the total amount of PCBs use in Japan is estimated approximately 53,000 tons.

As to 2.2.1 (the case in Lake Biwa) and to 2.2.2 (the case in Tsuruga Bay) another paper is prepared.

III REVISIONS OF ENVIRONMENTAL AND EFFLUENT STANDARDS ON MERCURY AND SETTINGS OF ENVIRONMENTAL AND EFFLUENT STANDARDS ON PCB

3.1 MERCURY

Revisions in the environmental quality standards and the waste water discharge standards relating to mercury and changes in the relevant measuring methods were enforced on Sept. 30, 1974. This action was done on a report presented to Director-General of Environment Agency in April, 1974, by the Central Council for Control of Environmental Pollution.

3.1.1 Reasons for Amendment

Under the old environmental standards on water quality,both alkyl mercury and total mercury were to "not detectable". But quantities of alkyl mercury smaller than 0.001 ppm could not be measured by gas chromatography and thin layer chromatograph-dithizon extraction absorptiometry, the procedures deemed appropriate when the standards were established in 1971. Likewise, there was a quantitative limit of 0.02 ppm for total mercury by dithizon extraction absorptiometry, the procedure practicable for administrative purposes in those days.

The "not detectable" level for alkyl mercury was set because of adverse effects on the health that result from eating heavily contaminated fish and shellfish over a long period of time and the difficulty of keeping it out of tap water.

In the case of total mercury, the intake from foods was subtracted from the intake level beyond which the metal would begin to accumulate in the human body, to figure out the intake from drinking water. The "not detectable" limit was instituted by considering the intake ascribable to drinking water, the difficulty of keeping the metal out of tap water, and a safety margin.

Both standards were not considered strict enough, but there was no choice but to be content with them because of the limited measuring capabilities at the time. It was also in consideration of these limitations that the effluent standards for mercury were set at the same level as the water quality standards when 10 times as much values as under the environmental quality standards were allowed for the discharge of other harmful substances.

Prompted by the following developments and in view of the urgent need to resolve the mercury pollution issue, the environmental quality standards and the waste water discharge standards relating to mercury were revised to update them: (1) Advances in analysis technology and the spread of more sophisticated analysing devices have made it possible to analyse low densities of mercury; (2) Provisional tolerance levels of mercury were fixed for fish and shellfish; (3) More data on environmentalpollution and fish and shellfish contamination have become available, permitting studies on the interrelationship between them.

Note 1: Technological advances— The measurable limit on total mercury by flameless atomic absorptiometry and on alkyl mercury by gas chromatography and thin layer chromatograph-flameless atomic absorptiometry has been improved to 0.0005 ppm, remarkably better than before.

Note 2: Provisional tolerance levels on fish and shellfish— The maximum permissible levels established provisionally by the Health and Welfare Ministry are 0.4 ppm in total mercury and 0.3 ppm in methyl mercury.

Note 3: Environmental pollution and fish and shellfish contamination— While surveys indicate that the mercury pollution of fish and shellfish has much more to do with bottom sediments than with water, it seems safe to say that if mercury levels in water are kept to the range of 0.005 ppm and 0.001 ppm, fish and shellfish living in the sea, lakes and marshes can stay well below the provisional limits.

3.1.2 Main Points of Revision

The revised standards relating to mercury are given below.

Revised Standards on Mercury

		Standard Value	Measuring Method
Environmental Quality Standards	Total Mercury	0.0005 ppm 1)	Flameless Atomic Absorptiometry
	Alkyl Mercury	Not Detectable 2)	Gas Chromatography and Thin Layer Chromatograph-Flameless Atomic Absorptiometry
Waste Water Discharge Standards	Total Mercury	0.005 ppm 3)	Flameless Atomic Absorptiometry
	Alkyl Mercury	Not Detectable 4)	Gas Chromatography and Thin Layer Chromatograph-Flameless Atomic Absorptiometry

Note 1: The standard value for total mercury stands for the annual mean value. It is to be eased to 0.001 ppm or less for rivers when there is no doubt that their pollution is spontaneous.

Note 2: The phrase "not detectable" applies to cases other than when the presence of alkyl mercury is detected by gas chromatography and thin layer chromatograph-flameless atomic absorptiometry.

Note 3: While the waste water discharge standard of

0.005 for total mercury is 10 times as much as the level allowed under the environmental quality standards, waste water released into a normal body of water is presumably diluted quickly to reach readings satisfying the environmental quality standards. Since the discharge limit applies to maximum levels of pollution, the average quality of released waste water will have to be better than 0.005 ppm.

Note 4: The discharge standard for alkyl mercury should be kept as strict as under the environmental quality standards because of its tendency to accumulate heavily in fish and shellfish.

3.2 PCB

The water quality panel of the Central Council for Control of Environmental Pollution has submitted a report on standards to fight PCB pollution to the Director General of the Environment Agency at the end of the year 1974. The panel's recommendations involved environmental water quality standards relating to PCBs, relevant waste water discharge standards, provisional standards for removal of PCB-contaminated sediments, and the analytical method to be employed.

The report was drafted in response to a request from the Environment Agency, which took the initiative to follow up on the earlier instituted measures to curb mercury pollution.

The agency's battle against PCB pollution has been based on decisions made by the PCB Pollution Countermeasures Council set up in April, 1972, and the Mercury Pollution Council created in June, 1973. The discharge of PCB-tainted waste water has been regulated on the basisof provisional guidelines presentedin July, 1972, while broad controls havebeen enforced on the production and use of

PCBs under the Chemical Substances Control Law.

In spite of these efforts, the PCB pollution of public bodies of water— resulting from the release of contaminated waste water from such facilities as the workshops of waste paper dealers— persists. Hence the agency's request for consideration of new anti-PCB measures.

Here are the main points of the report on PCB standards from the water quality panel.

3.2.1 Basic Principles

(1) The guiding principles in establishing the proposed sets of standards should be to keep down PCB levels in water and sediments so that the PCB level in fish and shellfish — whether accumulated directly from the contaminated environment or through the food chain— will not exceed 3 ppm as the provisional limit for foods. (See note)

(2) Allowances should be made for particular conditions of PCB pollution in diffront public bodies of water and for the accuracy of PCB measurements.

(3) Waste water discharge standards on PCBs should be established by considering the related environmental quality standards and the way PCB-contaminated waste water spreads when it is released from an outlet into a public body of water.

Note: The 3 ppm permissible limit on PCBs applies to the edible parts of fish and shellfish caught in inland seas, bays and other bodies of water. The provisional rule was set on the basis of recommendations made by the Food Sanitation Research Committee to the Health and Welfare Ministry on Aug. 14, 1972.

3.2.2 Consideration of Relevant Factors

(1) Concentration Ratio

The following formula should be used as the definition of the ratio of PCB concentration in fish and shellfish, a key factor in relation to the proposed sets of standards:

$$\text{PCB concentration} = \frac{\text{PCB density in edible parts of fish and shellfish}}{\text{PCB density in environmental water}}$$

Table 1 shows PCB concentration ratios obtained from experiments using this equation. The figures range from 5,607 to 8,582 and average 7,360. A proper ratio would be around 10,000, first because "hamachi" (young yellow-tails) and eels, the fish used in the experiments, are species more apt to accumulate PCBs and, second, because concentration ratios in cases where PCB densities actually pose a fish and shellfish contamination problem-- such densities would be a little lower than the range of one to five ppb, the PCB level of water in which the fish was kept— would be slightly higher than what the experiments showed. A third factor to be considered in this connection is the food chain.

(2) Measuring Methods for PCBs

Gas chromatography should be used to measure PCBs in enviromental water. The quantitative measurement limit under this method is 0.0005 ppm. As for PCBs in sediments, another measuring method is prescribed.

3.2.3 Environmental Quality Limit on PCBs

"Not detectable" is the recommendation on PCBs in environmental water, because, while the provisional PCB limit foods (3 ppm) and the proper PCB concetration ratio (10,000) suggest a ceiling of 0.0003 ppm, gas chromatography makes it possible to measure up to 0.0005 ppm. PCB densities in water should be obtained by the following formula:

$$\text{PCB density(ppm)}$$

$$= \frac{\text{PCB density in edible parts of fish and shellfish (ppm)}}{\text{PCB concentration ratio in fish and shellfish}}$$

$$= \frac{3 \text{ ppm}}{10{,}000} = 0.0003 \text{ ppm}$$

The setting of the "not detectable" environmental limit is not to be taken to mean that pollution up to 0.0005 ppm will be permitted. Currently clean water quality should be protected from degradation as much as possible.

3.2.4 Waste Water Discharge Standards

(1) Regulation Value— 0.003 ppm in PCBs

Based on the provisional PCB limit for foods and the appro- PCB concentration ratio, PCB densities in environmental water, are to be held to 0.0003 ppm or less. Since it is safe to assume that discharged waste water is diluted 10 times or more in a normal body of water, the regulation value is to be set at 0.003 ppm.

(2) Enforcement of Rule

A) The stiffened rule is not be enforced for a month after its promulgation to provide time in which to thoroughly inform industrial plants and other business establishments of the change. (Guidance has been exercised to keep PCB readings in discharged waste water below the earlier quantitative limit of 0.01 ppm under the above-mentioned provisional guidelines.)

B) Toilet paper manufacturing companies, using waste paper as material, are to be granted a one-year grace period in view of the particularly serious implication of the new rule for them in waste water disposal.

(3) Measuring during Grace Period

A) Targets for control of the quality of waste water, with 0.003 ppm as the permissible limit, are to be fixed for application to toilet paper producers during the grace period.

B) Administrative guidance is to be stepped up on waste paper collectors and manufacturers using such paper so that PCB-containing "no-carbon" copying paper will be handled separately from other kinds of junk, will not get mixed in production materials, and will be kept in strict custody once collected.

3.2.5 Provisional Standard For Removal of PCB- Contaminated Sediments

Considering statistical analysis on the relationship between the PCB pollution of fish and shellfish and PCB densities in the same areas based on national environmental surveys, sediments with a reading of around 10 ppm require removal.

The present dredging technology should make it feasible to bring the contamination level down to about 10 ppm. The removal standard is thus to be set at 10 ppm.

The imposition of tougher standards should be considered for bodies of water where the 10 ppm level is deemed inadequate to arrest the worsening PCB pollution of fish and shellfish.

IV PROVISIONAL STANDARDS FOR REMOVAL OF CONTAMINATED SEDIMENTS

4.1 MERCURY-CONTAMINATED SEDIMENTS

4.1.1 For Sea Area

Provisional standards for removal of mercury-contaminated sediments should be obtained by the following formula:

$$C = 0.18 \times \frac{\Delta H}{j} \times \frac{1}{s} \text{ (ppm)}$$

Here, C : Value of mercury density of sediments

ΔH: Mean value of the difference of sea levels between ebb and flow (m)

j : Dissolving ratio according to "the Method of

Survey for Bottom Sediment"

s : Safety factor

1) 10 for sea areas in which no fishing activities are performed.
2) 50 for sea areas in which fishing activities are performed, and in the case that the ratio of the catch for fish and shellfish which intake sediments and organisms sticking to sediments to the total catch is approximately less than ½.
3) 100 for sea areas in which fishing activities are performed, and in the case that the ratio of the catch for fish and shellfish which intake sediments and organisms sticking to sediments to the total catch is approximately exceeding than ½.

   Also, it is allowed that an adoption of stricter value of safety factor according to the special conditions, e.g., habit of diet.

   Here, "fish and shellfish which intake sediments and organisms sticking to sediments" are sorts of lobsters, crabs etc.

4.1.2 For Rivers and Lakes

In the case of rivers and lakes, sediments with a reading of over 25 ppm require removal. However, in the case of estuaries which are strongly affected by ebb and flow, it should be applied correspondingly in the case of sea areas, on the contrary in the case of sea areas which are affected strongly by the coastal current, it should be applied correspondingly in the case of rivers and lakes.

4.2 PCB-CONTAMINATED SEDIMENTS

As stated in the paragraph 3.2.5.

Fourth US/JAPAN Conference
on
Sewage Treatment Technology
Paper No. 3

STUDIES ON ADVANCED WASTE TREATMENT

October 24, 1975
Cincinnati, Ohio

Ministry of Construction
Japanese Government

STUDIES ON ADVANCED WASTE TREATMENT

1. **Current and Future Studies on Advanced Waste Treatment in Japan** 116
   M. Kashiwaya, PWRI, Ministry of Construction

2. **Outline of Research Activities on Advanced Waste Treatment in Public Works Research Institute, Ministry of Construction** 120
   M. Kashiwaya, PWRI, Ministry of Construction

3. **Direct Filtration of Secondary Effluent** 124
   M. Kashiwaya, PWRI, Ministry of Construction

4. **Metal Salts Precipitation** 156
   S. Ando, S. Kyosai and M. Kashiwaya, PWRI, Ministry of Construction

5. **Lime Precipitation and Recovery of Calcium Carbonate** 205
   S. Kyosai and K. Murakami, PWRI, Ministry of Construction

6. **Carbon Adsorption and Regeneration of Waste Carbon** 227
   S. Ando and K. Murakami, PWRI, Ministry of Construction

7. **Breakpoint Chlorination** 282
   H. Watanabe and K. Murakami, PWRI, Ministry of Construction

8. **Phosphate Removal in an Activated Sludge Facility by Alum Addition** 295
   T. Annaka, K. Kobori and K. Murakami, PWRI, Ministry of Construction

9. **Pilot Plant Studies of Phosphorus Removal from Secondary Effluent to Protect Lake Biwa** 316
   A. Sugiki, Japan Sewage Works Agency

CHAPTER 1. CURRENT AND FUTURE STUDIES ON ADVANCED WASTE TREATMENT IN JAPAN 117

1. CURRENT AND FUTURE STUDIES ON ADVANCED WASTE TREATMENT IN JAPAN

In recent years in Japan, so many experiments of pilot plant scales for advanced waste treatment have come to be carried out. These are roughly classified into the following three cases.

i) A case sponsored by the Central Government and assisted by local government

ii) A case conducted by local government independently or in cooperation with manufactures

iii) A case conducted by a manufacture alone

Advanced waste treatment now under study in Japan can be classified as to objectives as follows.

i) Experiment for upgrading the secondary effluent from the activated sludge process

- Major studies include: study of the effects of coagulant dose into aeration tank, study of effects of coagulant dose on biological treatment, and study on the selection of filter type for direct filtration of secondary effluent and selection of their media sizes and conponents.

ii) Experiment on tertiary treatment purposes for phosphorus removal from the secondary effluent

- So many pilot-scale experiments on chemical-sedimentation process using lime or metal salts as coagulants have been carried out.

  Purposes of the studies are: improvement of settling efficiency, improvement of phosphorus removal efficiency, improvement of organic matter reduction efficiency, and thickening and dewatering of the sludge.

  Experiments on recovery of used coagulant from settled sludge have not yet been reported.

iii) Experiments for the removal of residual organic matter from secondary effluent

- Experiments on granular activated carbon adsorption of secondary effluent or effluent from chemical-sedimentation have been reported one after another. Also, study on the regeneration for spent carbon is also prevailing.

  Granular activated carbon adsorption process has been practiced for the treatment of organic type industrial wastes. There are many factories with a large-scale waste water treatment facility equipped with a regenerator for spent carbon.

  Oxidation of organic matter of waste water by ozonation is rarely so long as the pilot plants are concerned, because the electric costs are so expensive in Japan.

iv) Experiments on biological nitrofication and denitrofication process.

- Experiments of three-stage biological nitrofication and denitrofication has been carried out with a pilot plant annexed to a small-scale sewage treatment facility.

This is because the secondary effluent from the small-scale sewage treatment facility is discharged into irrigation channel and nitrogen in the effluent hampers the growth of rice plant.

Strong opposition has been made by the farmers against the pollution of their paddy fields with nitrogen.

High concentration of nitrogen in the supernatant developed from the night soil digestion facility is also warned against by the farmers.

For this reason, demonstration plants for biological nitrofication and denitrofication were installed at several places and are now in operation.

v) Removal of ammonia by physico-chemical process

- Experiments on ammonia stripping, ion exchange of ammonium by making use of natural zeolite, and breakpoint chlorination have been conducted at several places each. Also, an experiment in which NOx in stack gas is reacted upon ammonia in sewage in order to vent ammonia into the open air in the form of N_2 gas is going to be started on a pilot plant scale by a private company.

  There has not been any trial for recovering ammonia in sewage in the form of industrial raw material or fertilizer.

vi) Experiments on the removal of inorganic substance in sewage

- In Japan, water shortage is expected to take place in future in Kanto and Kansai and other developed areas.

  Also, ground depletion areas (ground subsidence) have been reported increasingly as a result of overpumping of ground water.

  Against this backdrop, move toward using as industrial water the tertiary effluent of sewage has been intensified. Reflecting the trend, a fair number of pilot plants using reverse osmosis or electrodialysis process have been operated. Also, many are struggling for the development of membrane to be used for such processes.

The Central Government-subsidized public works projects concerning the tertiary treatment in FY 1975 (April 1975 ~ March 1976) are only two; Minami-Tama Sewage Treatment Plant under the Tamagawa Basin-Wide Sewerage Works, Tokyo Metropolitan Government, and Tone Sewage Treatment Plant under the Jonan Basin-Wide Sewerage Works, Ibaragi Prefecture.

Since the public waters in Japan have been seriously polluted, the Central Government and many local governments alike are strongly urged to upgrade the effluent discharged from sewage treatment plants.

Such being the circumstances, the construction of tertiary treatment facilities will be pushed forward at most of sewage treatment plants.

Japan is short of the under-ground natural resources, and a narrow, overpopulated country. A population of more than 114 million live on the narrow country. The national living mainly relies on imports of natural resources which are converted into industrial products for export.

Some 300 sewage treatment plants installed in the past have all counted on the biological process using conventional or modified (including conpact types) activated sludge process mainly.

Most of the sewage treatment plants to be constructed from now on will be adopted conventional activated sludge process as the secondary treatment process.

Herewith, the sewage treatment system for the future should be the design that is based on the saving of resources and energy, and should also be able to recover materials contained in the sewage as much as we can.

The studies on the advanced waste treatment in Japan will therefore be guided and promoted by the Central Government, local governments and private manufactures for the development of new technology meeting the said requirements with paramount importance attached to the scarcity of resources and energy.

CHAPTER 2. OUTLINE OF RESEARCH ACTIVITIES ON ADVANCED WASTE TREATMENT IN PUBLIC WORKS RESEARCH INSTITUTE, MINISTRY OF CONSTRUCTION 121

2. OUTLINE OF RESEARCH ACTIVITIES ON ADVANCED WASTE TREATMENT IN PUBLIC WORKS RESEARCH INSTITUTE, MINISTRY OF CONSTRUCTION

Public Works Research Institute, Ministry of Construction, has carried out some experiments of pilot plant scale at the Shitamachi Sewage Treatment Plant, Yokosuka, and the Toba Sewage Treatment Plant, Kyoto, and with part of the existing facilities at the Nishiyama Sewage Treatment Plant, Nagoya.

In these field experiments, engineers and chemists of the Water Pollution Control Division, Public Works Research Institute and of the Sewerage and Sewage Purification Bureaus of respective cities have joined efforts for engineering, construction and operation control of pilot plants or actual facility as well as for the collection of data. In the meantime, the Water Quality Control Division of the Public Works Research Institute has undertaken a laboratory study for the purpose of designing the pilot plant and complementing the data obtained in the field studies.

As reported at the 3rd U.S.-Japan Conference on Sewage Treatment Technology held in Tokyo on February, 1974, the members of the Joint Working Group on Advanced Waste Water Treatment Technology which is composed of the engineers and chemists from the Ministry of Construction and eight cities have also participated in the laboratory test of the chemical sedimentation process using lime and metal salts, and have collected the data.

At the pilot plant located in the Shitamachi Sewage Treatment Plant in Yokosuka, chemical sedimentation process has been examined in two trains; one using lime as a coagulant and the other using metal salts. In the lime-using chemical sedimentation process test, quick-lime has been put to continuous slaking to make coagulant. In the metal salts-using chemical sedimentation test, various aluminum salts and iron salts have been tried as coagulants.

The experiments with the pilot plant at the Shitamachi Sewage Treatment Plant, Yokosuka, have been made for the comparison between various coagulants as to the efficiency of chemical sedimentation process for removal of phosphorus and organic matter, study on the method of improving settling efficiency by trying various coagulants and settling attachment, and tests for thickening and dewatering of sludge containing coagulants.

Also, the Yokosuka Municipal Government has conducted a study for converting the effluent from the sewage treatment plant into industrial water by using the effluent of the pilot plant. Recently, this study has been continued by applying the granular activated carbon adsorption process and reverse osmosis process.

Of the items reported hereunder, those concerning the lime sedimentation process and metal salts sedimentation process use the experimental data mainly obtained from the operation of this pilot plant.

With the pilot plant located at the Toba Sewage Treatment Plant, Kyoto, direct filtration, granular activated carbon adsorption and breakpoint chlorination have been studied. At present, a chemical sedimentation tank using metal salts as coagulants is under construction to add to the existing facilities, and the experiment is scheduled to be started in September this year.

The Toba Sewage Treatment Plant in Kyoto is one of the most largest sewage treatment plants in Japan, and is featured in that its influent BOD concentration mixed with supernatant of sludge treatment process is as high as 200 to 350 mg/lit.

The application of the secondary effluent of the Toba sewage treatment plant to the pilot plant for study purposes is advantageous in that experimental data for the case where sewage is flowing into the sewage treatment plant at a design rate can directly be obtained. In general, BOD concentration of the raw sewage running into the sewage treatment plants are expected to rise more and more in the future in our country.

In view of these facts, the tertiary pilot plant studies at the Toba Sewage Treatment Plant in Kyoto will have a great significance on the preparation of manual for engineering, operation and maintenance of the tertiary facilities to be installed at sewage treatment plants in Japan.

The experiments with the pilot plant at the Toba Sewage Treatment Plant, Kyoto, have embraced studies on the development and upgrading to a practical level of instrumentations and automatic control system and for the tertiary treatment facilities.

The down flow gravity type filter and the chemical sedimentation tank using metal salts as coagulants which is going to enter upon operation in September this year are designed to freely control the filtration flow rate of the filter and the in flow of the chemical sedimentation tank on current signals from the program transmitter.

Also, the composite samplers of the influent and effluent of the filter and chemical sedimentation tank are designed to operate on the signals from the same program transmitter.

Of the items reported hereunder, those concerning direct filtration of secondary effluent, activated carbon adsorption process, and the regeneration of spent carbon as well as the breakpoint chlorination process are based on the data acquired from this pilot plant.

Also, the data from this pilot plant are used in "Automatic Water Quality Measurement for Wastewater Treatment."

The Nishiyama Sewage Treatment Plant, Nagoya, is treating domestic sewage only with a design sewage flow of 30,000 m^3/d.. Raw sewage of 20,000 to 22,000 m^3 per day is flowed into the treatment at present.

The plant is composed of two primary sedimentation tanks, two aeration tanks and three final sedimentation tanks (of which two have been used for this experiment).

As sludge treatment is not carried out in this treatment plant, supernatants from the sludge treatment facility are not sent into the primary or secondary treatment process.

As regards one train in the treatment plant, a experiment of dosing aluminum sulphate into the aeration tank has been made. Another train has been used as a control and operated under conventional activated sludge process. In the Nishiyama Sewage Treatment Plant, these investigations have been carried out: purification effects of aluminum sulphate dose into aeration tank, influence of aluminum sulphate dose on the organisms and their biological activities in the activated sludge process, and measurement of sludge productions in the primary sedimentation tanks and final sedimentation tanks by means of magnetic flowmeters and ultrasonic sludge concentration meters. It is to be added by the way that one train of automatic backwashing filter comprising 8 compartment vessels is scheduled to be put in operation in September this year.

Of the items reported hereunder, those concerning "phosphate removal in an activated sludge facility by alum addition" are based on the data obtained from the Nishiyama sewage treatment plant, Nagoya. Also, "Study on the treatment and disposal of sludge" uses data acquired by the investigations at this plant.

Since the beginning of FY 1975, the Public Works Research Institute has been conducting laboratory studies for nitrofication and denitrofication of municipal sewage. Especially, the subject of nitrofication study is first stage nitrofication by using the primary and secondary treatment facilities only of those sewage treatment plants which are operated on the conventional activated sludge process. If the laboratory study is successful, it will be amplified into field experiments at the Nishiyama Sewage Treatment Plant.

CHAPTER 3. DIRECT FILTRATION OF SECONDARY EFFLUENT

3.1 Effects on the Filtration and Washing of the Ratio of the Depth of Anthracite to the Total Depth of Media in the Down-Flow Type Gravity Dual Media Filter 125

3.2 Effects on the Filtration and Washing of the Difference between Dual Media and Mixed Media in the Down-Flow Type Gravity Declining Filtration 126

3.3 Effects of the Sizes of Media in the Down-Flow Gravity Filter on the Filtration and Washing 127

3.4 Comparison of Filtration Effects between Constant Flow-Rate Filtration and Declining Flow-Rate Filtration in the Down-Flow Type Gravity Dual Media Filtration 128

3.5 Comparison of Filtration Effects between the Variable Flow-Rate Filtration and Declining Flow-Rate Filtration in the Down-Flow Type Gravity Dual Media Filtration 129

3.6 Merits and Demerits of Up-Flow Filter 130

3. DIRECT FILTRATION OF SECONDARY EFFLUENT

The Kyoto Pilot Plant now jointly experimented with by the Ministry of Construction and the Kyoto Municipal City has three down-flow type gravity filters and one up-flow type filter, each measuring 1.0 m in width, 1.2 m in depth and 3.75 m in height.

Various field studies concerning direct filtration of secondary effluent have been carried out by using these four pilot filters, and some of the findings were reported at the 3rd U.S.-Japan Conference on Sewage Treatment Technology held in Tokyo on February 1974. Later, the pilot filters were modified partly. The data appearing in this report are obtained from the experiments following the flow chart shown in Fig. 3.1.

What is changed from the system reported previously is the installation of a various flow-rate pattern transmitter whose output signal is used to control one of the down-flow type gravity filter. The Roots pump connected to the effluent pipes of this filter is feedforward controlled by the current signal from the flow pattern transmitter, and at the same time feedback controlled by the current signal from the automatic level gauge on the triangle weir of the effluent measuring tank.

At the end of August this year, two other down-flow type gravity filters were equipped with a Roots pump and an electric circuit for the same control operation as above.

As regards the up-flow type filter, which was found as explained later having troubles with the experiment, a separate test has been made independent of the down-flow type gravity filters.

3.1 EFFECTS ON THE FILTRATION AND WASHING OF THE RATIO OF THE DEPTH OF ANTHRACITE TO THE TOTAL DEPTH OF MEDIA IN THE DOWN-FLOW TYPE GRAVITY DUAL MEDIA FILTER

Part of the results concerning this investigation was reported at the 3rd U.S.-Japan Conference on Sewage Treatment Technology held in Tokyo on February 1974. Since then, the investigation has progressed, and the following conclusions are reached.

The effective size and coefficient of uniformity of the media used in the test are as shown in Table 3.1. The listed media were filled up into the filter No. 1 (type I) and the filter No. 2 (type II) shown in Table 3.2.

The test was conducted on the declining filtration only; both filters got started at the same time, and when a maximum filtration head loss of 3.0 m was attained, they were stopped. After the other filters completed filtration, they were washed in turn, and were restarted to resume filtration test.

a. As shown in Table 3.3, the filtration run length at filtration flow rate of 180 m^3/m^2.d. to 420 m^3/m^2.d. was about twice as long in type II as in type I. It was found by means of a head loss meter set in the media that anthracite layer has a larger capability of holding suspended solids in the secondary effluent than silica sand layer. It is therefore more advantageous, in the light of the filtration run length, the larger the thickness of anthracite layer is.

b. Type I and type II are compared in Table 3.4 with respect to filtrate quality and their removals.
   There is no significant difference between the two.

c. The initial head losses of clean media were compared. For a filtration flow rate of 360 m^3/m^2. d., for example, the filter arrangement of type I caused an initial head loss of 54 cm, while the arrangement in type II recorded 36 cm. The difference was 1.5 in ratio.

d. It was confirmed that with the most suitable expanded bed ratio set at 20% in the back washing, the back washing rate for type II can be o.71 m^3/m^2. min. as against 0.87 m^3/m^2. min. for type I.
   The larger the expanded bed ratio becomes, the larger is the difference in back washing rate between the two.

e. It takes either type 10 to 12 min. to complete back washing.
   For either type, surface washing is indispensable.

f. Washing volume/filtrate volume ratio reached 2.3 to 3.2% in case of type I filter arrangement, but was 1.1 to 2.0% in type II filter arrangement.

It is therefore concluded that the depth of the anthracite layer be maximized. At any rate, the depth of the anthracite layer should be more than 60% of the total depth if it is used in the direct filtration down-flow type gravity filter.

The maximum ratio of anthracite layer depth to the total media depth, above 65%, has yet to be studied.

3.2 EFFECTS ON THE FILTRATION AND WASHING OF THE DIFFERENCE BETWEEN DUAL MEDIA AND MIXED MEDIA IN THE DOWN-FLOW TYPE GRAVITY DECLINING FILTRATION

Like item 3.1, this was also reported with some results at the 3rd U.S.-Japan Conference on Sewage Treatment Technology held in Tokyo on February 1974.

Since then, additional experiments have been carried out, and the following conclusions are reached.

The effective size and coefficient of uniformity of the media used in the experiments are as given in Table 3.1. The media were filled up in filter No. 2 (type II) and filter No. 3 (type III). Sizes of the media shown in Table 3.1 were brand new ones. When preparing the mixed media, they were brought into the filter in several hauls, and each time back washing was carried to remove fine grains. Thus, the effective size and coefficient of uniformity were changed accordingly. In type II, back washing was carried out after the media were filled up totally; namely, fine grains over the surface of the anthracite layer alone were removed.

a. As shown in Table 3.3, the filtration run length at filtration flow rate of 180 m^3/m^2. d. to 420 m^3/m^2. d. tended to become a little longer in type II arrangement than in type III.
   The time history of the readings of the head loss meter set inside the filter in order to evaluate the capacity of obstruction of suspended solids in the secondary effluent remained almost common to both types.

b. The filtrate quality and their removals are compared between type II and type III in Table 3.4. As is clear, type II was a little better in filtrate quality than type III.
   In type III, the lowermost layer was formed with garnet sand having an effective size of 0.32 mm., but it was not found the effect of the garnet layer by the readings of the head loss meter.

c. The initial head losses of clean media were compared. At filtration flow rates of not exceeding 240 m^3/m^2 . d., there was little or no change in the initial head loss. At 360 m^3/m^2 . d., type III showed some 3 cm larger loss than type II.

d. When the most suitable expanded bed ratio of media in the back washing was set at 20%, both types revealed a back washing flow rate of 0.71 m^3/m^2 . min. For both types, 10 to 12 min. of back washing time was necessary. The surface washing was indispensable for both types.

e. In most cases, the washing volume/filtrate volume ratio was a thought smaller in type II than in type III, which would reflect the fact that type II took a little longer filtration run length.

It was found that the filtration effect of the mixed media is a little inferior to the dual media type using the same sizes of anthracite and silica sand. It still remains obscure whether this is due to poor selection of size or ascribable to the way of arranging the mixed media layers.

It is conjectured that so long as the direct filtration of secondary effluent is concerned, there would not be developed any remarkable difference between the dual media and mixed media even if an improvement is achieved.

3.3 EFFECTS OF THE SIZES OF MEDIA IN THE DOWN-FLOW TYPE GRAVITY FILTER ON THE FILTRATION AND WASHING

After comparative study on filtration performance of three types shown in Table 3.2, tests were carried out on different media with filters No. 1 and No. 2. The effective size and coefficient of uniformity of new media are given in Table 3.5. The constitution of the filter bed of the filters No. 1 and No. 2 in shown in Table 3.6. As regards the declining filtration, comparison study was made with respect to the difference in the size of media between the filter No. 3 (type III) and filter No. 2 (type IV). The results are as follows.

a. The difference in head loss curve between the two, time variations of filtration flow rate, and change of filtrate turbidity are shown in Figs. 3.2 and 3.3. The difference in filtration run length in the declining filtration was larger with an initial filtration flow rate of 420 m^3/m^2 . d. than 180 m^3/m^2 . d. The decrement of the filtration flow rate with time was also larger with 420 m^3/m^2 . d. than 180 m^3/m^2 . d.

b. Irrespective of the filtration flow rate, type III showed a little better filtrate turbidity than type IV as shown in Figs. 3.2 and 3.3. The same held true with suspended solids, BOD and COD_{Mn} as given in Table 3.7 and 3.8., though the difference was very small.

c. In type III, the back washing flow rate necessary for attaining a expanded bed ratio of 20% was 0.71 m^3/m^2 . min., while type IV required 1.03 m^3/m^2 . min. The time necessary for the back washing was almost the same for both.

d. Both types showed no difference in the washing volume/filtrate volume ratio when operated at a filtration flow rate of 180 m^3/m^2 . d.
However, type IV showed smaller ratio than type III, the larger the filtration flow rate became.

It is therefore concluded that the effective size of anthracite should preferably be more than 1.6 mm. just as with type IV. This is particularly the case when the operation is carried out at a high filtration flow rate.

3.4 COMPARISON OF FILTRATION EFFECTS BETWEEN CONSTANT FLOW-RATE FILTRATION AND DECLINING FLOW-RATE FILTRATION IN THE DOWN-FLOW TYPE GRAVITY DUAL MEDIA FILTRATION

A comparative study was conducted on type IV (declining flow-rate filtration) and type V (constant flow-rate filtration). The results are as follows.

a. For the initial filtration flow rate, 180 m^3/m^2 . d., shown in Fig. 3.4, there was little or no diffference in the total head loss during filtration.
In the case of declining flow rate filtration, however, the filtration flow-rate was decreased to 120 m^3/m^2 . d. when the total head loss reached 3.0 m.

b. For the initial filtration flow-rate, 420 m^3/m^2 . d., shown in Fig. 3.5, filtration run length caused a difference of some 12 hrs. It became longer when the declining flow rate filtration was practised. In the declining flow rate filtration, however, the declining of filtration flow-rate was noticeable; when the total head loss was 3.0 m in the constant flow rate filtration, the declining flow rate filtration had a reduced filtration flow rate of 280 m^3/m^2 . d.
When the total head loss in the declining flow rate filtration reached 3.0 m, the filtration flor rate became 240 m^3/m^2 . d. or 57% of the initial filtration flow-rate.

c. As regards the turbidity, suspended solids, BOD and COD_{Mn} listed in Table 3.7 both types showed little or no difference in the filtrate quality. In the case of direct filtration, the declining flow-rate filtration was not effective in improvement of filtrate quality any more than the constant flow-rate filtration.

d. As regards the washing volume/filtrate volume ratio, the constant flow rate filtration was better than the declining flow-rate filtration.

The employment of declining flow rate filtration in the direction of secondary effluent faces various difficulties in the planning of the filter. Considering the fact that it is by no means excellent than the constant flow rate filtration, its employment is quite meaningless.

3.5 COMPARISON OF FILTRATION EFFECTS BETWEEN THE VARIABLE FLOW-RATE FILTRATION AND DECLINING FLOW-RATE FILTRATION IN THE DOWN-FLOW TYPE GRAVITY DUAL MEDIA FILTRATION

A flow pattern transmitter was installed for the filter No. 1 in order to give a typical flow pattern of secondary effluent in the sewage treatment plant upon which to operate the filter. This was type V. A comparison study was made between type V and type IV in declining flow rate filtration. The flow pattern curve applied to the testing of type V was based on the measurements taken from the Toba Sewage Treatment Plant. The flow pattern transmitter generated a pattern for a filtration flow rate of 300 $m^3/m^2 . d$. When the mean filtration flow rate was to be changed from the above value, the flow pattern was changed in proportion to the deviation. In the event that the filtration flow-rate was larger than 300 $m^3/m^2 . d$., the difference between the maximum and the minimum flow rate became large, while the difference became smaller if the rate smaller than 300 $m^3/m^2 . d$.

The results obtained with the mean filtration flow rates of 180 $m^3/m^2 . d$., 240 $m^3/m^2 . d$., 300 $m^3/m^2 . d$., 360 $m^3/m^2 . d$., and 420 $m^3/m^2 . d$. are shown in Figs. 3.6, 3.7, 3.8, 3.9 and 3.10, respectively.

The results are as follows.

a. The filtration run length becomes smaller in the variable flow-rate filtration than in declining flow rate filtration.
The difference however is small in case of the mean filtration flow rate is 180 $m^3/m^2 . d$. and 420 $m^3/m^2 . d$. For other rates, the difference is remarkable. Generally, the smaller the inflow of suspended solids into the filter, the smaller the difference in filtration run length between variable flow-rate filtration and declining flow rate filtration.

b. The total filtration head loss in the variable flow rate filtration changes with the media's capacity to detain suspended solids. Irrespective of the value of mean filtration flow rate, the total filtration head loss increases sharply with increase in the filtration run length. This tendency is more stronger the larger the mean filtration flow-rate becomes.

c. The filtrate quality in the variable flow rate filtration is a little inferior to that in the declining flow-rate filtration, though the difference is very small as shown in Table 3.8. This tendency also applies to those other than turbidity given in Figs. 3.6 through 3.10.

d. As regards the washing volume/filtrate volume ratio, there is seen little or no difference between the variable flow rate filtration and declining flow rate filtration.

All these suggest high feasibility of variable flow-rate filtration for the direct filtration of the secondary effluent, signifying that the variable flow-rate filtration is worth further examination. Since the influent of the sewage treatment plant varies largely at all hours, installation of a regulating reservoir for direct filtration of secondary effluent would be necessary if constant or declining flow-rate filtration is practiced at a sewage treatment plant. For this reason, the practice of variable flow-rate filtration within the range of not degrading the filtrate quality is quite an economical technique for polishing the secondary effluent.

So far this kind of filtration technique has not been taken up seriously. If the variable flow rate filtration is found to be able to raise a required efficiency, the polishing of the secondary effluent will be feasible at any sewage treatment plant economically.

With this in mind, the experiments with the down-flow type gravity filter on the direct filtration of secondary effluent will be continued with priority given to the experiments of variable flow-rate filtration.

3.6 MERITS AND DEMERITS OF UP-FLOW FILTER

The up-flow filter has been developed in the Luton Sewage Treatment Plant and the Blackbird Sewage Treatment Plant in Britain.

This type of filter permits the use of influent directly for the back washing. Also, it dispenses with post aeration as dissolved oxygen concentration in the filtrate becomes high.

For this reason, an up-flow filter having a filter area of 1.2 m^2 was installed at the Kyoto Pilot Plant, and has been experimented with. The media size and depth are shown in Table 3.9. On top of the filter, a grid was placed for the purpose of suppressing the expansion of bed.

The results of experiments with this filter and considerations are as follows.

a. As shown in Table 3.10, the filtration run length for the maximum filtration head loss to reach 3.0 m was 20 to 30% longer compared with the declining flow rate down-flow filter (type II). Whereas the up-flow filter is to be operated at a constant flow-rate filtration, the up-flow filter under consideration got directly connected with a influent booster pump as shown in Fig. 3.1, whereby the filtration flow-rate was reduced during operation. The decrement of the filtration flow rate was less than one-fifth the declining flow rate down-flow type filter's. (See Fig. 3.11)

b. The total filtration head loss curve of the up-flow filter under normal operating conditions is shown in Fig. 3.11.
This filter caused leakage locally in its filter bed, and the increase of the total head loss was stopped accordingly. Naturally, the suspended solids concentration in the filtrate rose up. An example of this phenomenon is shown in Fig. 3.12.
This kind of leakage phenomenon happens for certain irrespective of the filtration flow-rate. The causes are still unknown.
Morigasaki Sewage Treatment Plant, Tokyo, is installed with two up-flow filters, each having a filter area of 30 m^2, which have been used for testing. These filters also have experienced leakage. The Sewage Bureau of the Tokyo Metropolitan Government has been pushing forward a survey for the improvement of washing technique and for the feasibility of preventing the bed leakage by limiting the total filtration head loss to 2.8 m.

c. As regards the turbidity, suspended solids, BOD and COD_{Mn} listed in Table

3.11. Up-flow filter showed little or no difference as compared with down-flow declining filter in the filtrate quality.

d. In the up-flow filter, the bed is put to cleaning by air agitation and back washing in combination. The experiments with the up-flow filter in the Kyoto Pilot Plant disclosed that the cleaning is not sufficient if air agitation and back washing alone are counted upon. For this reason, the following method was tried.

Draining of filter bed – Air agitation (4 min.) – Rest (2 min.) – Air agitation (4 min.) – Water pulsating washing (20 sec.) – Rest (1 min.) – (10 repetitions of water pulsating washing and rest in combination) – Water back washing (10 min.) – Water level depression in filter bed – Water back washing (10 min.) – Start of filtration

Although the above cleaning procedure has been tried, it still is uncertain whether the media arrangement is warrantable to always keep the filter bed clean and to prevent leakage phenomenon. There is a fear of causing a great quantity of overflow of media from the washed water trough as the entire filter bed comes afloat in the pressurized water unless the filter bed is monitored all the way from the beginning of water pulsating washing. In the experiments with the up-flow filter at the Kyoto Pilot Plant, less importance was attached to the flotation of the filter bed, and there has been experienced overflow of a great amount of media from the washed water trough several times.

e. At the Morigasaki Sewage Treatment Plant, Tokyo, investigations have been made as to the cleaning method of up-flow filter bed.

As a result, the following technique is proposed as best.

Draining of filter bed – Air agitation (6 min.) and water back washing (10 min.) in combination (the final four min. for water back washing only) – Air agitation (6 min.) and water back washing (12 min.) in combination (the final six min. for water back washing only) – Rest (5 min.) – Start-up of filtration

It is claimed that the rest prepared in the final stage is indispensable for the stabilization of media and consequently for the prevention of leakage phenomenon.

f. The same cleaning method as in the Morigasaki Sewage Treatment Plant has not been tried in the Kyoto Pilot Plant as the combined use of air agitation and water back washing necessitates a large scale modification of electric control circuit used for the up-flow filter.

As the past experiments suggests, it is evident that sudden application of "air agitation and water back washing" can cause a great deal of overflow of media from the washed water trough as the media are buoyed.

The overflow trouble due to water pulsating washing or back washing may be peculiar only to small-scale filters like one in the Kyoto Pilot Plant. In a large scale up-flow filter, the media may not be buoyed uniformly at the time of water washing. Once in a while, the Morigasaki Sewage Treatment Plant, Tokyo, has been processing only a limited quantity of influent, and the concentration of suspended solids in the secondary effluent is less than 10 mg/lit..

On the other hand, the concentration of suspended solids in the secondary effluent of the Toba Sewage Treatment Plant is Kyoto is as high as 25 to 35 mg/lit. Some fear that if secondary effluent containing high ratio of suspended solids like this is directly filtered, the media separation would become difficult because of adhesive power developed by suspended solids between media particles.

These doubts will be clarified by further investigations.

g. The up-flow filter in Kyoto Pilot Plant requires some 1 hr. for washing of filter bed. On the other hand, the Morigasaki's is some 40 min. Then, the down-flow filter requires less than 20 min. even if surface washing and back washing are carried out in combination.

The washing water volume for the up-flow type filter is about 2.2 to 3 times as large as that for the down-flow type. It is evident that the down-flow filter is superior to up-flow filter when viewed from the washing volume/filtrate volume ratio.

As discussed in the foregoing, the up-flow filter has various problems to solve before it will be put to practice. For the purpose of direct filtration of secondary effluent, further investigations are necessary.

The authors have continued experiments with the up-flow filter believing in the fact that it permits the use of influent for washing purposes, which is hardly expected of the down-flow filter. The authors will continue efforts for refining the up-flow filter until it can stand practical use.

Table 3.1 Media Size Using Experiment of Down Flow Type Gravity Filters

Media	Effective size (mm)	Uniformity coefficient
Anthercite coal	0.95	1.48
Silica sand	0.64	1.50
Garnet sand	0.32	2.16

Table 3.2 Media Depth Using Experiment of Down Flow Type Gravity Filters (Type I, II and III)

Type of experiment / Used filter / Kinds of media	Type I	Type II	Type III
	No. 1	No. 2	No. 3
Anthercite coal	150 (mm)	625 (mm)	625 (mm)
Silica sand	850	375 (mm)	300
Garnnet sand	–	–	75
Total	1,000	1,000	1,000
Type of filtration	Declining	Declining	Declining

Table 3.3 Operational Data of Filtration Run Length of Down Flow Pilot Filters
(Max. Total Head Loss 3.0 meter) (hrs)

Types of experiment	Used filter	Items	Flow-rate ($m^3/m^2 \cdot d$)				
			180	240	300	360	420
Type I	Filter No. 1	Max.	62:00	54:75	50:30	36:25	47:67
		Min.	6:33	9:75	9:75	4:00	7:00
		Ave.	32:93	28:96	20:92	21:40	20:33
Type II	Filter No. 2	Max.	125:25	101:50	125:25	65:75	100:00
		Min.	11:25	7:50	21:75	4:50	12:50
		Ave.	71:55	53:17	48:07	38:80	50:00
Type III	Filter No. 3	Max.	129:00	94:00	113:50	57:75	94:00
		Min.	31:00	22:25	23:25	7:50	9:15
		Ave.	69:23	58:49	40:00	33:32	37:70

Table 3.4 Operational Data of Down Flow Pilot Filters (Water Quality of Influents and Effluents and Their Removals)

Items		Type of experiment	Type I					Type II					Type III				
		Used filter	Filter No. 1					Filter No. 2					Filter No. 3				
		Flow-rate ($m^3/m^2 \cdot d$)	180	240	300	360	420	180	240	300	360	420	180	240	300	360	420
Turbidity	Conc.	Inf.	8.4	6.6	6.9	7.8	7.8	8.0	6.5	6.9	8.4	8.1	7.9	6.6	6.8	7.9	7.7
	(mg/ℓ)	Eff.	2.9	2.3	2.5	3.1	5.5	3.7	2.8	2.9	3.2	5.3	4.1	2.8	3.1	3.2	5.8
	Removal (%)		65.5	65.2	63.8	60.0	29.5	53.8	56.9	60.0	64.4	34.6	48.1	57.6	54.4	59.5	24.8
Sus. solids	Conc.	Inf.	13.3	12.8	11.5	14.6	15.0	12.9	12.5	12.0	14.0	12.7	13.0	12.5	12.1	12.8	13.2
	(mg/ℓ)	Eff.	4.0	3.7	3.4	4.5	5.9	5.2	3.2	4.4	3.7	6.0	4.0	3.6	4.5	4.3	5.6
	Removal (%)		69.9	71.1	70.4	69.2	60.7	59.7	74.4	63.3	73.5	52.8	69.2	71.2	62.8	66.4	57.6
BOD	Conc.	Inf.	22.8	19.4	21.2	27.2	20.6	22.9	17.9	21.8	25.3	17.1	22.8	17.6	23.8	23.0	16.8
	(mg/ℓ)	Eff.	7.9	4.8	4.9	6.8	7.2	6.9	5.3	6.0	7.6	7.4	7.2	4.6	7.2	7.9	6.7
	Removal (%)		65.4	75.3	76.9	75.0	65.0	69.9	70.4	72.5	70.0	56.7	68.4	73.9	69.7	65.7	60.1
COD_{Mn}	Conc.	Inf.	17.0	15.1	14.5	16.1	15.0	17.3	15.0	15.7	16.2	15.3	16.7	15.1	15.7	15.9	13.9
	(mg/ℓ)	Eff.	14.5	11.8	12.1	12.5	12.6	14.5	12.1	12.1	13.0	12.9	15.8	12.8	12.9	12.9	13.1
	Removal (%)		14.7	21.9	16.6	22.4	16.0	16.2	19.3	22.9	19.8	15.7	5.4	15.2	17.8	18.9	5.8

Table 3.5 Media Size Using Experiment of Down Flow Type Filters

Media	Effective size (mm)	Uniformity coefficient
Anthercite coal	1.62	1.327
Silica sand	0.605	1.256

Table 3.6 Media Depth Using Experiment of Down Flow Type Gravity Filters (Type IV, Type V and Type VI)

Type of experiment	Type IV	Type V	Type VI
Used filter Kinds of media	No. 2	No. 1	No. 1
Anthercite coal	625 (mm)	625 (mm)	625 (mm)
Silica sand	325	325	325
Total	1,000	1,000	1,000
Type of filtration	Declining	Constant	Variable

Table 3.7 Operational Data of Down Flow Pilot Filters
(Water Quality of Influent and Effluent and Their Removals)

Types of Experiment			Type V					Type IV					Type III				
Used Filter			Filter No. 1					Filter No. 2					Filter No. 3				
Items / Flow-rate ($m^3/m^2 \cdot d$)			180	240	300	360	420	180	240	300	360	420	180	240	300	360	420
Turbidity	Conc. (mg/l)	Inf.	3.9	4.3	2.7	–	2.8	3.9	4.3	2.7	–	2.8	3.9	4.3	2.7	–	2.8
		Eff.	2.1	3.0	2.2	–	2.1	2.5	3.1	2.5	–	2.3	2.6	3.2	2.6	–	2.3
	Removal (%)		46.2	30.2	18.5	–	25.0	35.9	27.9	7.4	–	17.9	33.3	25.6	3.7	–	17.9
Sus. Solids	Conc. (mg/l)	Inf.	6.4	4.5	3.0	–	3.2	6.4	4.5	3.0	–	3.2	6.4	4.5	3.0	–	3.2
		Eff.	2.5	4.0	3.0	–	2.1	2.7	3.8	(3.1)	–	2.5	3.0	4.1	2.6	–	2.2
	Removal (%)		60.9	11.1	0	–	34.4	57.8	15.6	–	–	21.9	53.1	8.9	13.3	–	31.3
BOD	Conc. (mg/l)	Inf.	14.9	6.0	6.0	–	20.1	14.9	6.0	6.0	–	20.1	14.9	6.0	6.0	–	20.1
		Eff.	6.1	4.4	2.8	–	12.6	4.6	4.5	3.0	–	13.1	5.9	5.9	3.5	–	12.9
	Removal (%)		59.1	26.7	53.3	–	37.3	69.1	25.0	50.0	–	34.8	60.4	1.7	41.7	–	35.8
COD_{Mn}	Conc. (mg/l)	Inf.	15.7	17.9	14.0	–	17.7	15.7	17.9	14.0	–	17.7	15.7	17.9	14.0	–	17.7
		Eff.	14.6	17.4	(16.2)	–	16.3	14.3	16.2	(16.7)	–	15.6	14.2	16.8	(19.7)	–	16.9
	Removal (%)		7.0	2.8	–	–	7.9	8.9	9.5	–	–	11.9	9.6	6.2	–	–	4.5
Remarks			Constant Flow-rate					Declining Flow-rate					Declining Flow-rate				

Table 3.8 Operational Data of Down Flow Pilot Filter
(Water Quality of Influent and Effluent and Their Removals)

Types of Experiment			Type VI					Type IV					Type III				
Used Filter			Filter No. 1					Filter No. 2					Filter No. 3				
Items	Flow-rate ($m^3/m^2 \cdot d$)		180	240	300	360	420	180	240	300	360	420	180	240	300	360	420
Turbidity	Conc. (mg/l)	Inf.	5.4	5.6	4.4	8.2	6.2	5.4	5.6	4.4	8.2	6.2	5.2	5.6	4.4	8.2	6.2
		Eff.	3.5	4.2	2.7	5.3	4.3	3.5	4.2	3.1	5.1	4.5	3.5	4.0	2.8	4.9	4.1
	Removal (%)		35.2	25.0	38.6	35.4	30.7	35.2	25.0	29.6	37.8	27.4	32.7	28.6	36.4	40.2	33.9
Sus. Solids	Conc. (mg/l)	Inf.	7.5	7.1	5.5	9.8	9.0	7.5	7.1	5.5	9.9	9.0	7.5	7.1	5.5	9.7	9.0
		Eff.	3.9	5.2	4.0	6.6	7.5	3.8	4.8	4.2	6.0	6.5	3.5	4.1	3.7	6.3	6.4
	Removal (%)		48.0	26.8	27.3	32.7	16.7	49.3	32.4	23.6	39.4	27.8	53.3	42.3	32.7	35.1	28.9
BOD	Conc. (mg/l)	Inf.	19.3	12.7	20.7	17.5	16.5	19.4	12.7	20.7	17.6	16.5	20.1	12.7	20.7	17.5	16.5
		Eff.	6.0	6.5	10.0	7.3	9.4	5.9	5.8	9.5	7.6	8.9	5.8	5.7	9.6	6.7	9.2
	Removal (%)		68.9	48.8	51.7	58.3	43.0	69.6	54.3	54.1	56.8	46.1	71.1	55.1	53.6	61.7	44.2
COD_{Mn}	Conc. (mg/l)	Inf.	25.0	21.8	20.7	20.5	24.2	24.6	21.8	20.7	20.3	24.2	24.9	21.8	20.7	20.4	24.2
		Eff.	22.7	17.7	16.4	16.7	22.7	21.8	18.5	18.4	17.2	(27.3)	20.7	18.2	17.1	16.5	23.7
	Removal (%)		9.2	18.8	20.8	18.5	6.2	11.4	15.1	11.1	15.3	–	16.9	16.5	17.4	19.1	2.1
Remarks			Variable Flow-rate					Declining Flow-rate					Declining Flow-rate				

Table 3.9 Media Size and Depth for Up-flow Filter

Items \ Size and depth	Media size		Depth (mm)
	Effective size (mm)	Uniformity coefficient	
Filter media (Silica sand)	1.16	1.33	1,550
	2.11	1.31	
Sand for media support	2~ 3		300
Gravel for media support	10~15		250
	20~30		100

Table 3.10 Operational Data of Filtration Run Length of Both Up-flow Filter and Down Flow Filter

(hrs.)

Types of Experiment	Used Filter	Items	Flow-rate (m^3/m^2·d)			
			180	240	300	360
Up-flow Filter	Filter No. 4	Max.	131.00	105.00	124.45	55.50
		Min.	54.25	20.50	20.00	23.50
		Ave.	91.45	68.80	55.90	48.50
Down-flow Filter (Type II)	Filter No. 2	Max.	125.25	101.50	125.25	65.75
		Min.	11.25	7.50	21.75	4.50
		Ave.	71.55	53.17	48.07	38.80

Table 3.11 Comparison of Filtrate Quality of Both Up-Flow Filter and Down-Flow Filter

Type of filter			Up-flow filter				Down-flow filter (Controlled)				
Filter number			No. 4				No. 2				
Initial flow-rate ($m^3/m^2 \cdot d$)			180	240	300	360	180	240	300	360	420
Turbidity	Conc. (mg/ℓ)	Inf.	8.0	6.1	6.9	8.3	8.0	6.5	6.9	8.4	8.1
		Eff.	3.4	3.4	3.2	3.1	3.7	2.8	2.9	3.2	5.3
	Removal (%)		57.5	44.3	53.6	62.7	53.8	56.9	60.0	64.4	34.6
Sus. solids	Conc. (mg/ℓ)	Inf.	13.0	11.4	12.2	13.0	12.9	12.5	12.0	14.0	12.7
		Eff.	4.0	4.0	5.0	4.6	5.2	3.2	4.4	3.7	6.0
	Removal (%)		69.2	64.9	59.0	64.6	59.7	74.4	63.3	73.5	52.8
BOD	Conc. (mg/ℓ)	Inf.	22.8	17.8	23.4	21.4	22.9	17.9	21.8	25.3	17.1
		Eff.	6.8	5.7	6.8	6.8	6.9	5.3	6.0	7.6	7.4
	Removal (%)		70.2	70.0	70.9	31.8	69.9	70.4	72.5	70.0	56.7
COD_{Mn}	Conc. (mg/ℓ)	Enf.	16.7	14.5	16.0	14.0	17.3	15.0	15.7	16.2	15.3
		Eff.	15.3	12.9	13.1	12.7	14.5	12.1	12.1	13.0	12.9
	Removal (%)		8.4	11.0	18.1	9.3	16.2	19.3	22.9	19.8	15.7

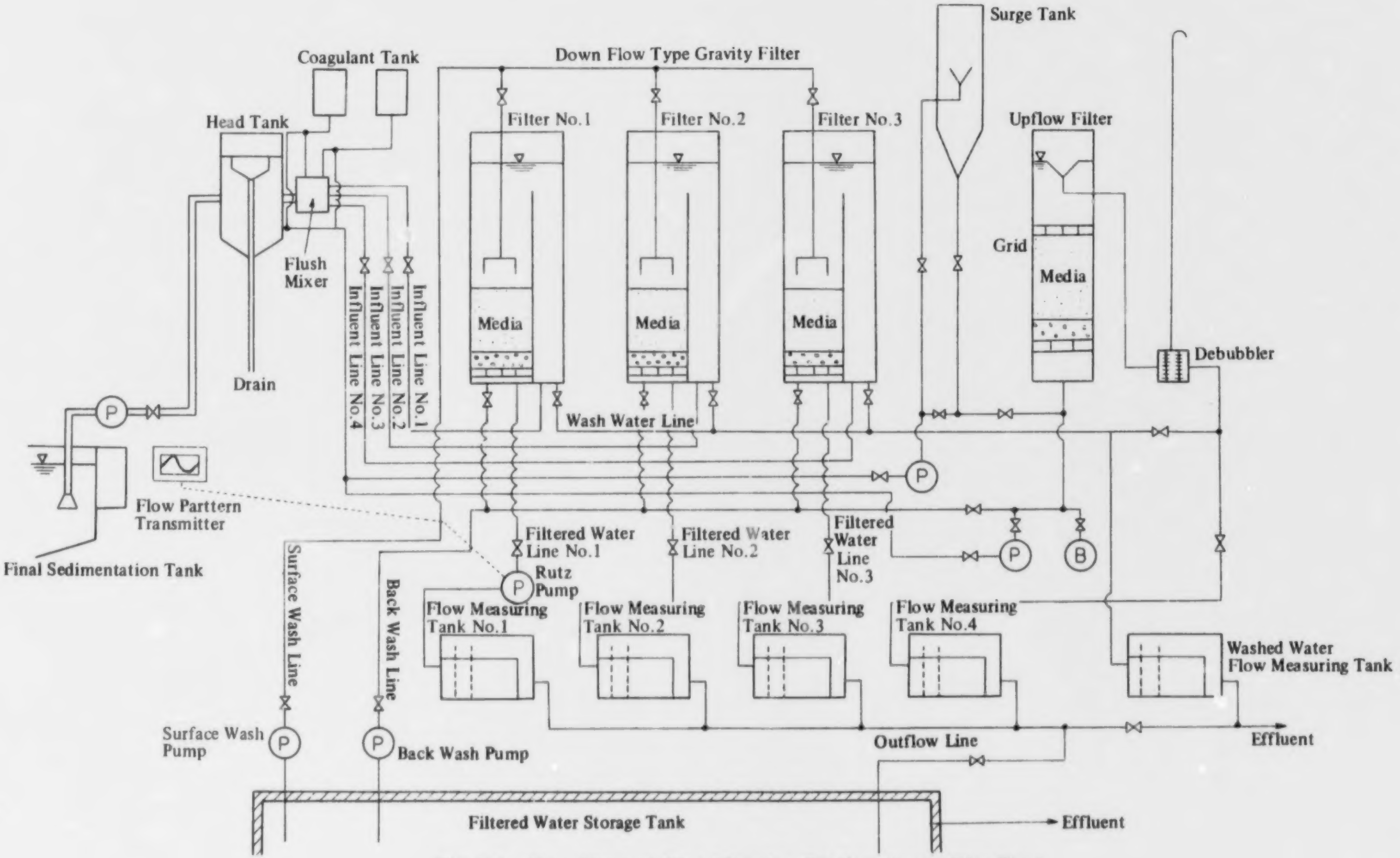

Coagulant Tank
Down Flow Type Gravity Filter
Surge Tank
Head Tank
Filter No.1
Filter No.2
Filter No.3
Upflow Filter
Grid
Media
Media
Media
Media
Flush Mixer
Influent Line No.1
Influent Line No.2
Influent Line No.3
Influent Line No.4
Drain
Debubbler
Wash Water Line
P
P
P
B
Flow Parttern Transmitter
Final Sedimentation Tank
Filtered Water Line No.1
Filtered Water Line No.2
Filtered Water Line No.3
Rutz Pump
Flow Measuring Tank No.1
Flow Measuring Tank No.2
Flow Measuring Tank No.3
Flow Measuring Tank No.4
Washed Water Flow Measuring Tank
Surface Wash Line
Back Wash Line
Surface Wash Pump
Back Wash Pump
Outflow Line
Effluent
Filtered Water Storage Tank
Effluent

Fig. 3.1 Flow Chart of Pilot Filtration Facility, Kyoto Pilot Plant

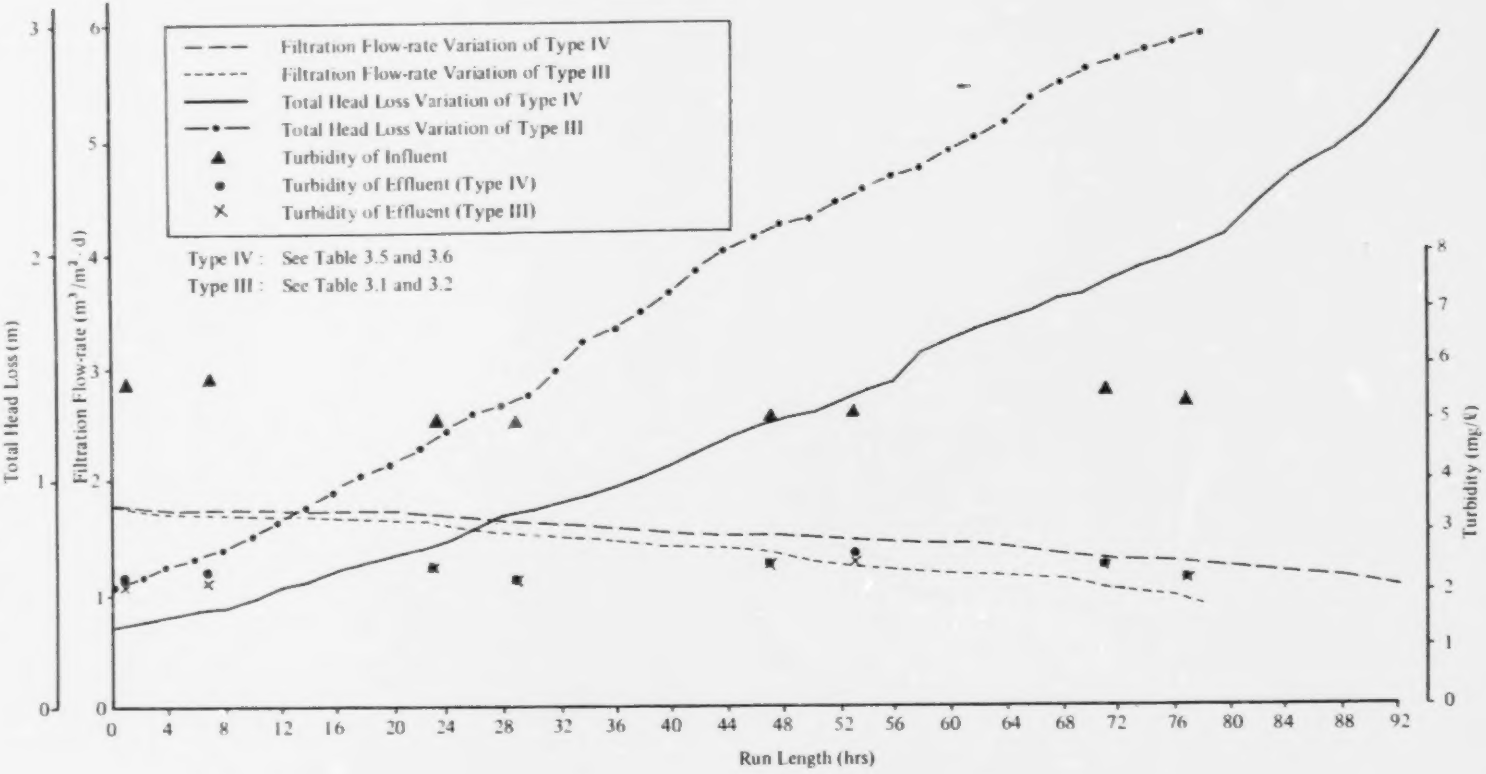


Fig. 3.2 Effect of Media Size in Down-flow Declining Dual Media Filter on Total Head Loss, Flow Rate, and Turbidity Variations

(Initial Filtration Flow-rate 180 $m^3/m^2 \cdot d$)

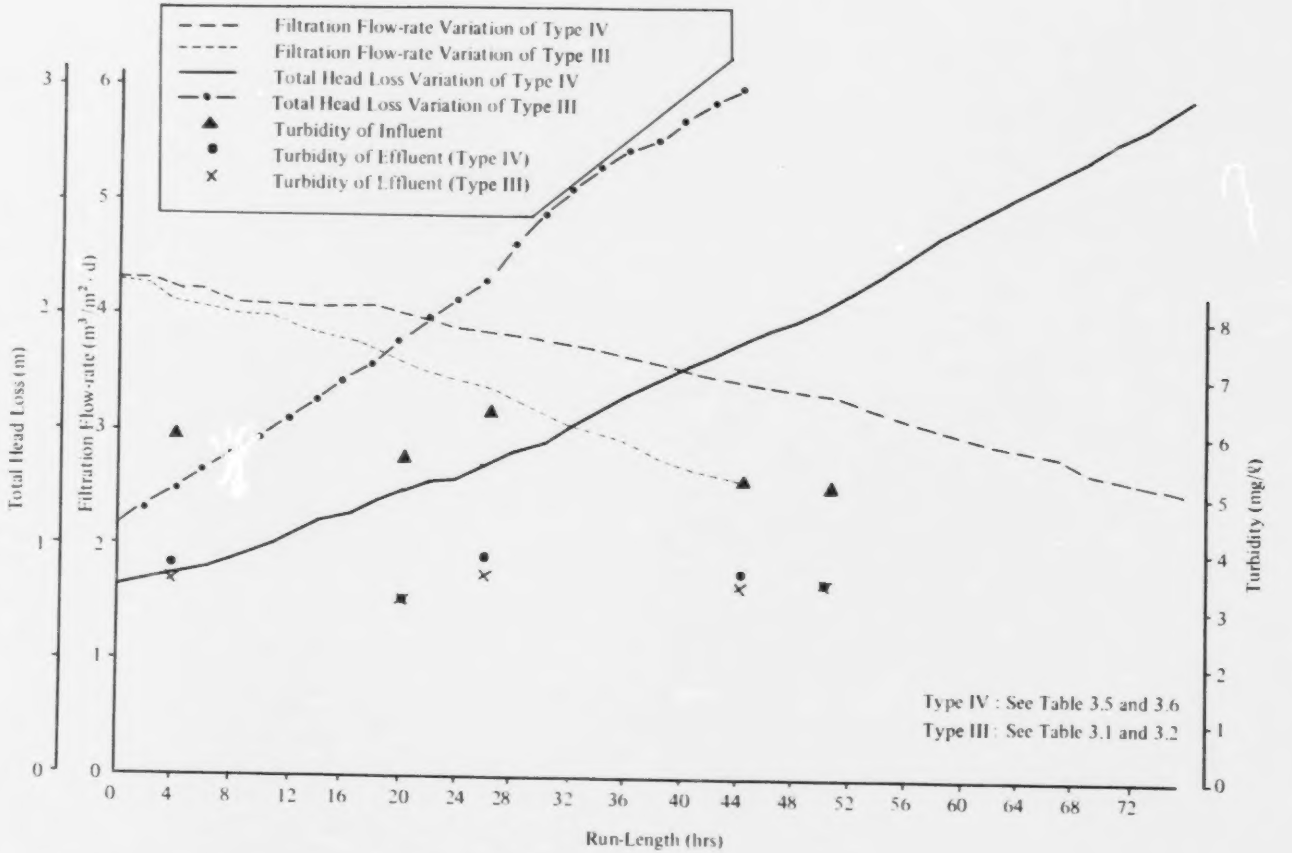


Fig. 3.3 **Effect of Media Size in Down-flow Declining Dual Media Filter on Total Head Loss, Flow Rate, and Turbidity Variations**

(Initial Filtration Flow-rate 420 $m^3/m^2 \cdot d$)

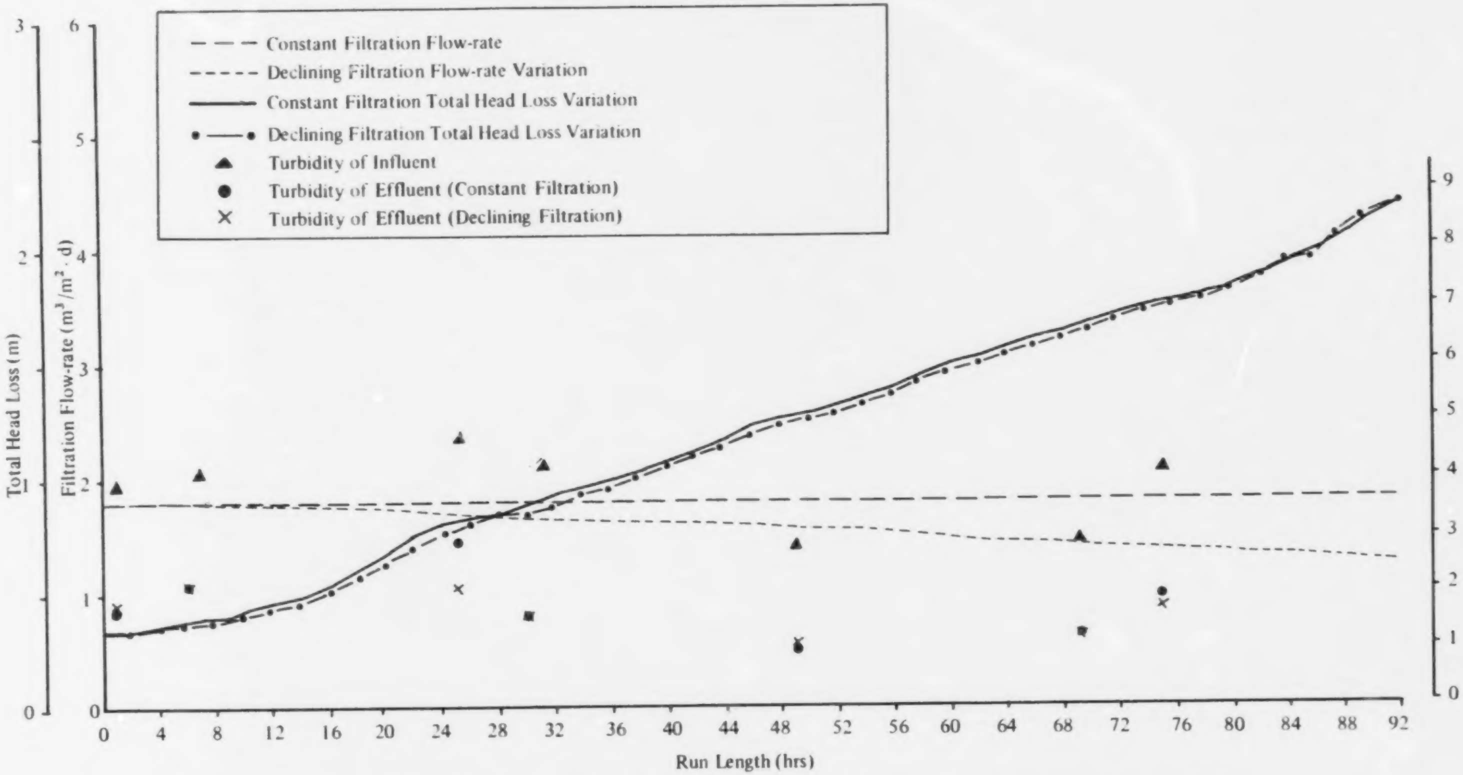


Fig. 3.4 Comparison of Constant Flow-rate and Declining Flow-rate Filtrations on Total Head Loss, Flow-rate, and Turbidity Variations in Down-flow Dual Media Filters
(Initial Filtration Flow-rate 180 $m^3/m^2 \cdot d$)

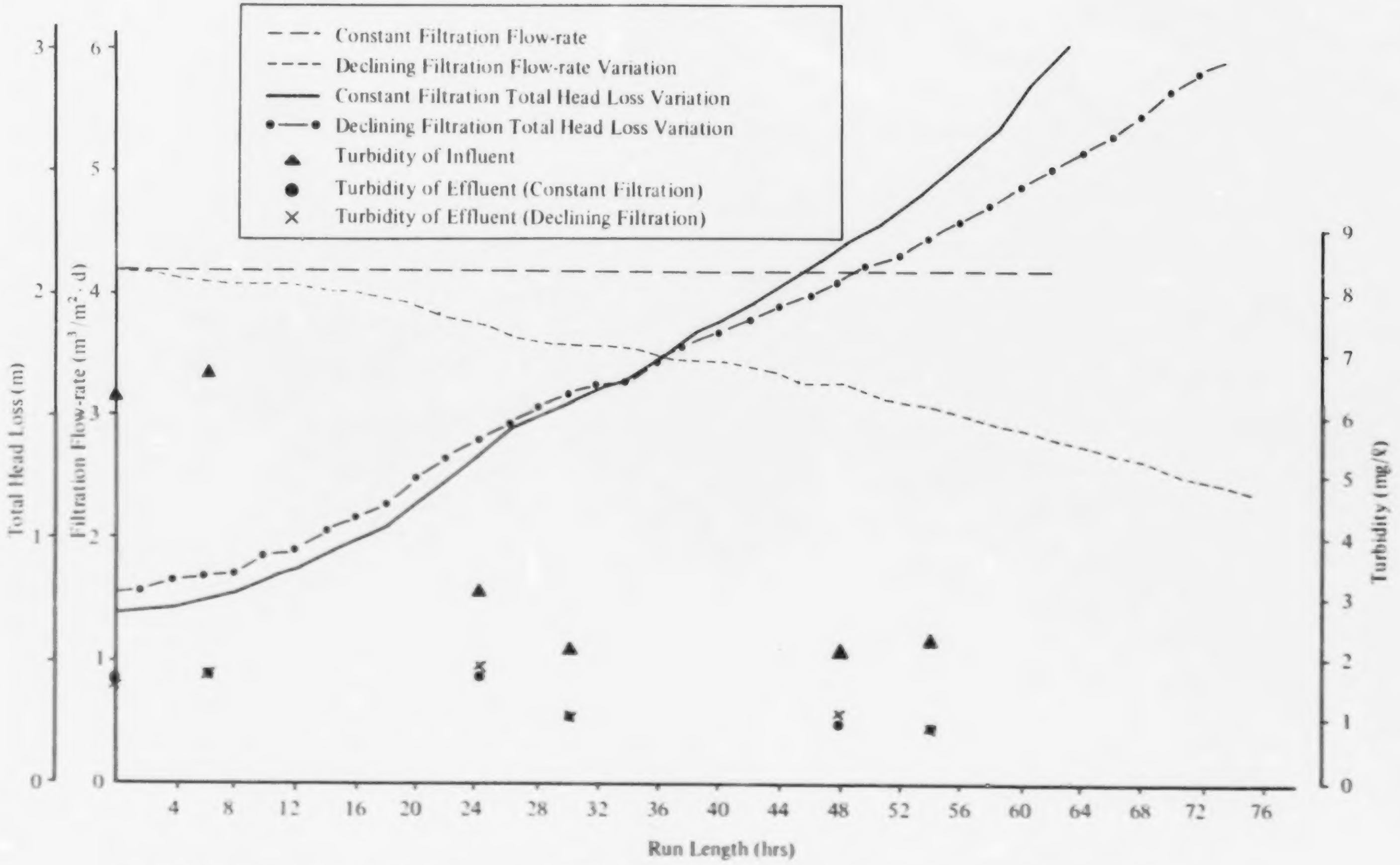


Fig. 3.5 Comparison of Constant Flow-rate and Declining Flow-rate Filtrations on Total Head Loss, Flow-rate, and Turbidity Variations in Down-flow Dual Media Filters
(Initial Filtration Flow-rate 420 m^3/m^2 · d)

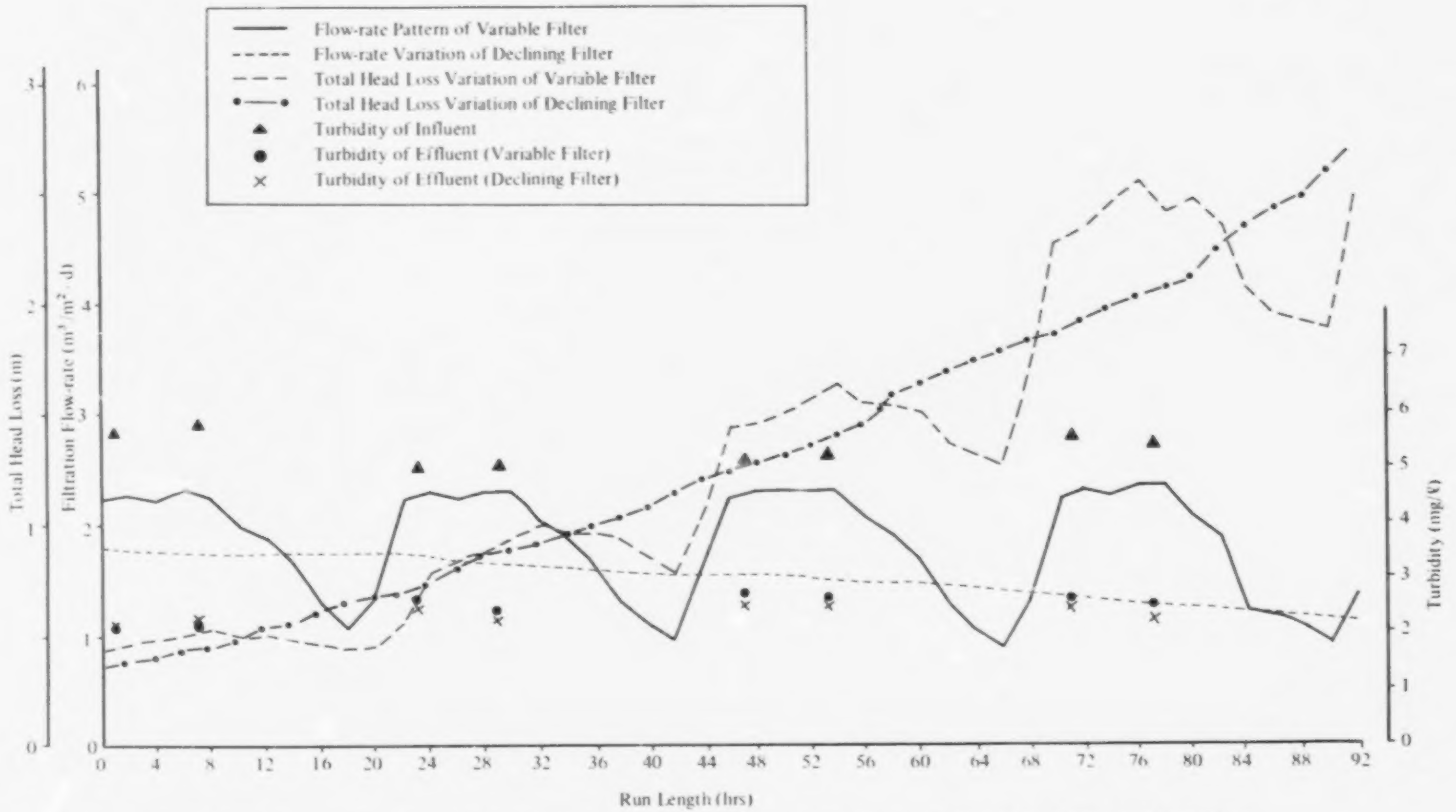


Fig. 3.6 Comparison of Variable Flow-rate and Declining Flow-rate on Flow-rate, Total Head Loss and Turbidity Variations in Down-flow Dual Media Filters

[Average Flow-rate of Variable Filter: Approx. 180 $m^3/m^2 \cdot d$]
[Initial Flow-rate of Declining Filter: 180 $m^3/m^2 \cdot d$]

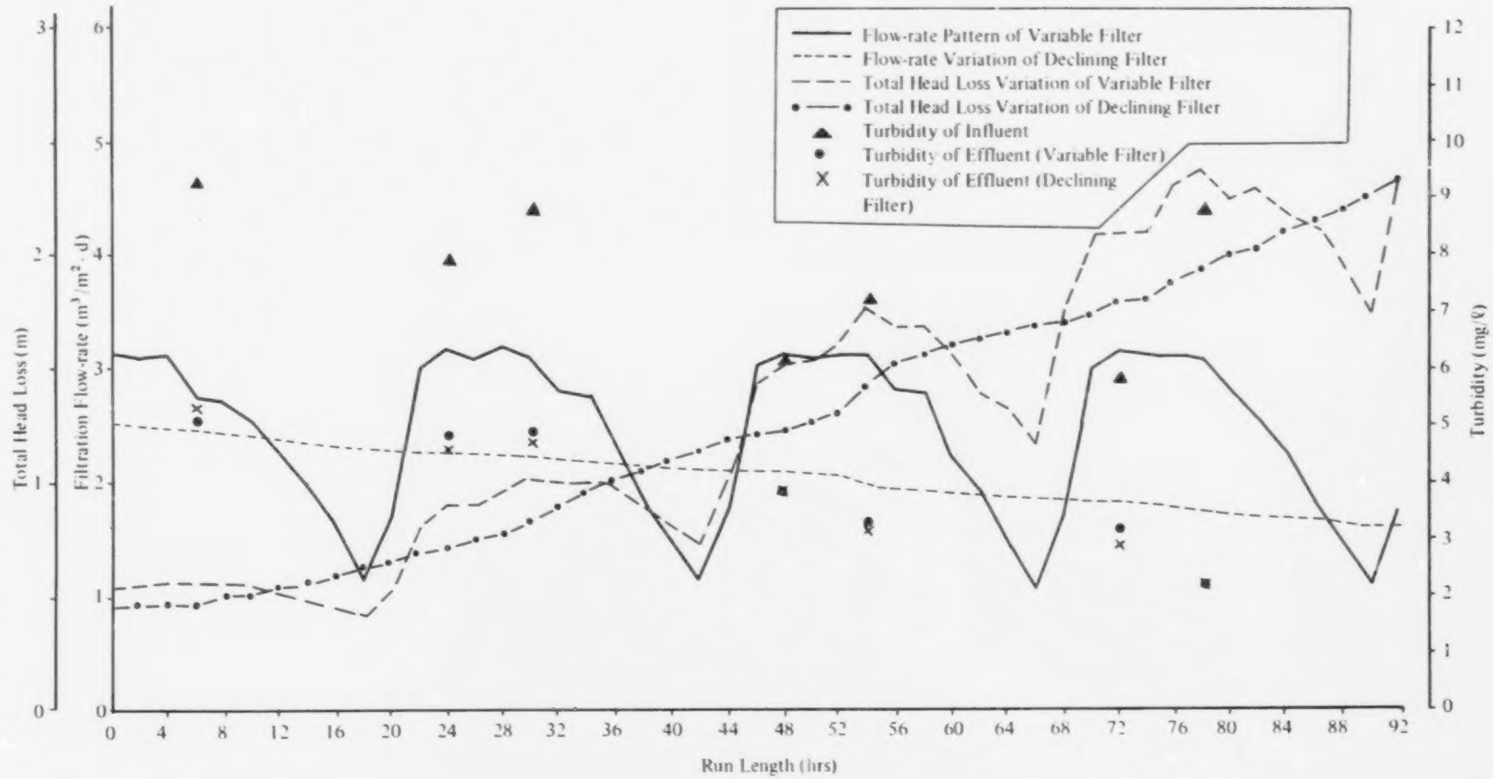


Fig. 3.7 Comparison of Variable Flow-rate and Declining Flow-rate on Flow-rate, Total Head Loss and Turbidity Variations in Down-flow Dual Media Filters

[Average Flow-rate of Variable Filter: Approx. 240 $m^3/m^2 \cdot d$
Initial Flow-rate of Declining Filter: 240 $m^3/m^2 \cdot d$]

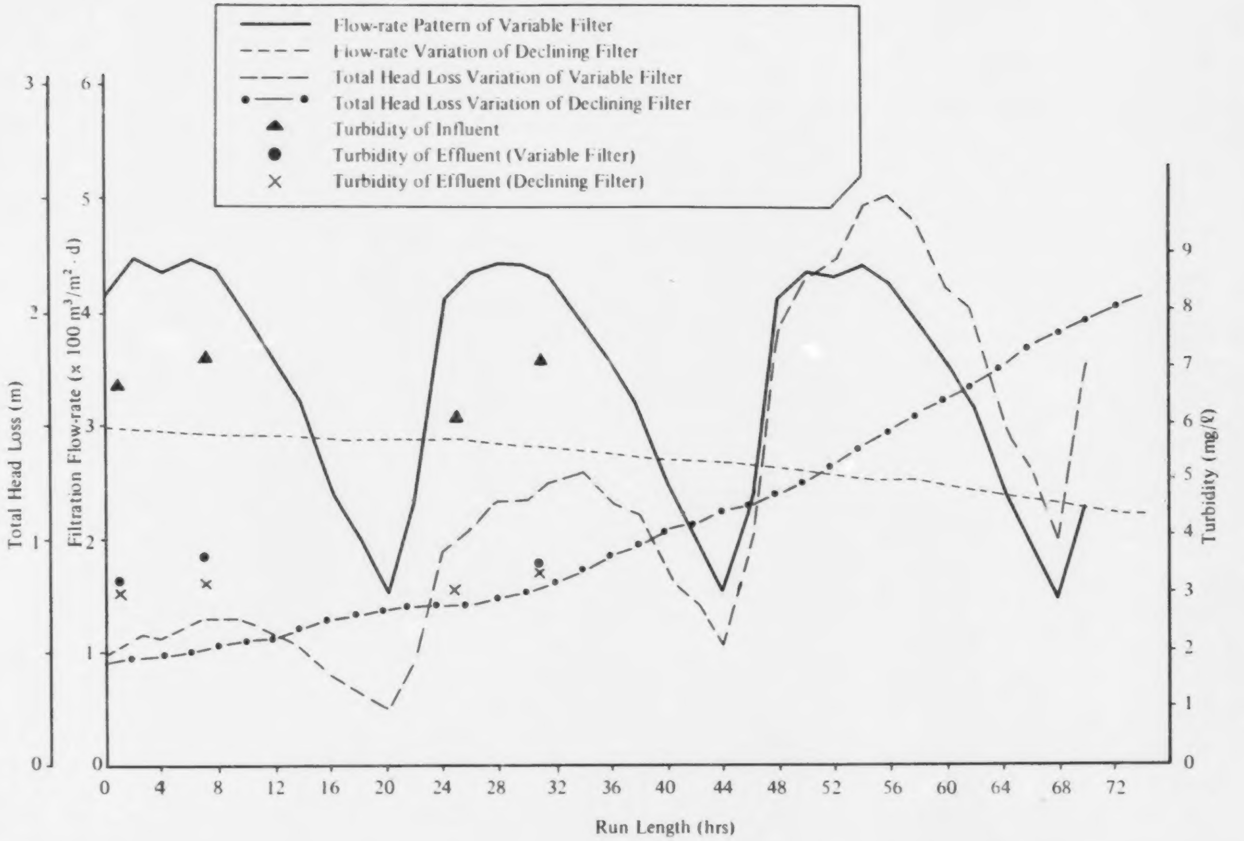


Fig. 3.8 Comparison of Variable Flow-rate and Declining Flow-rate on Flow-rate, Total Head Loss and Turbidity Variations in Down-flow Dual Media Filters

[Average Flow-rate of Variable Filter: Approx. 300 $m^3/m^2 \cdot d$
Initial Flow-rate of Declining Filter: 300 $m^3/m^2 \cdot d$]

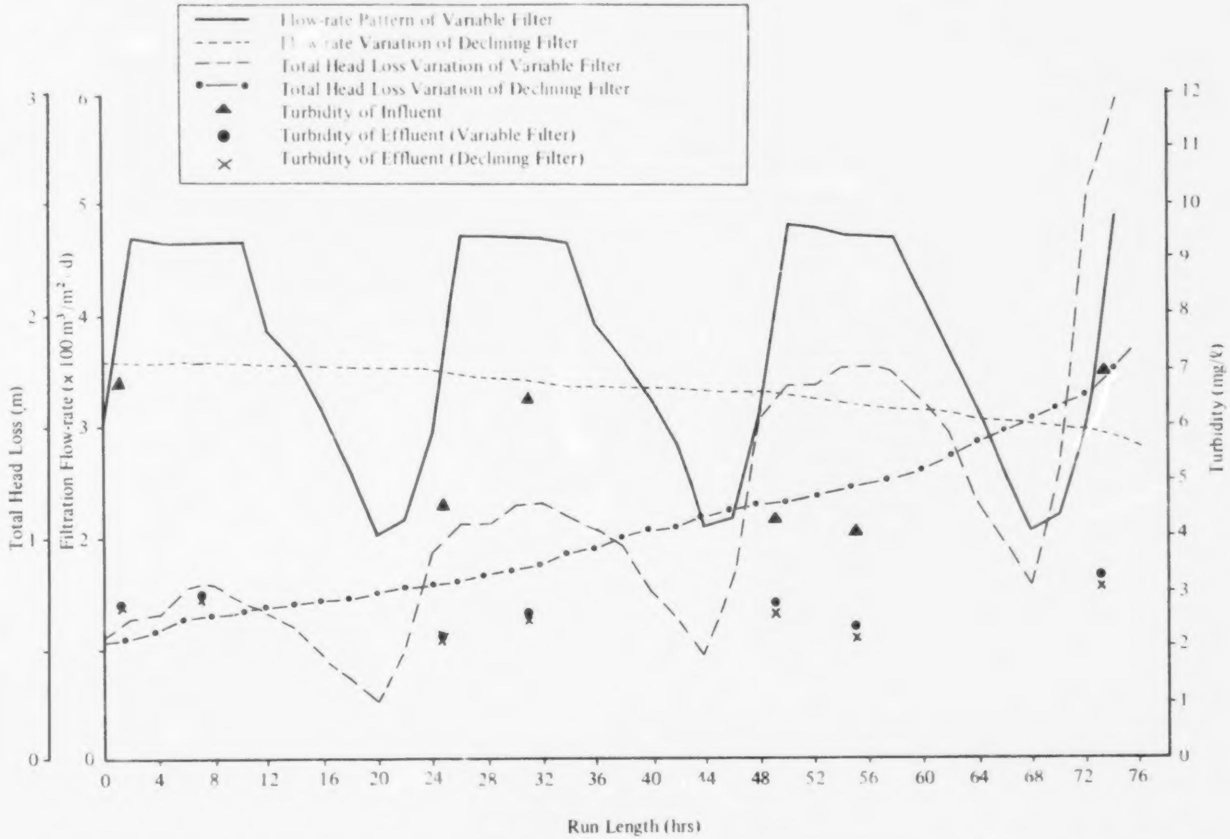


Fig. 3.9 **Comparison of Variable Flow-rate and Declining Flow-rate on Flow-rate, Total Head Loss and Turbidity Variations in Down-flow Dual Media Filters**

[Average Flow-rate of Variable Filter: Approx. 360 $m^3/m^2 \cdot d$
Initial Flow-rate of Declining Filter: 360 $m^3/m^2 \cdot d$]

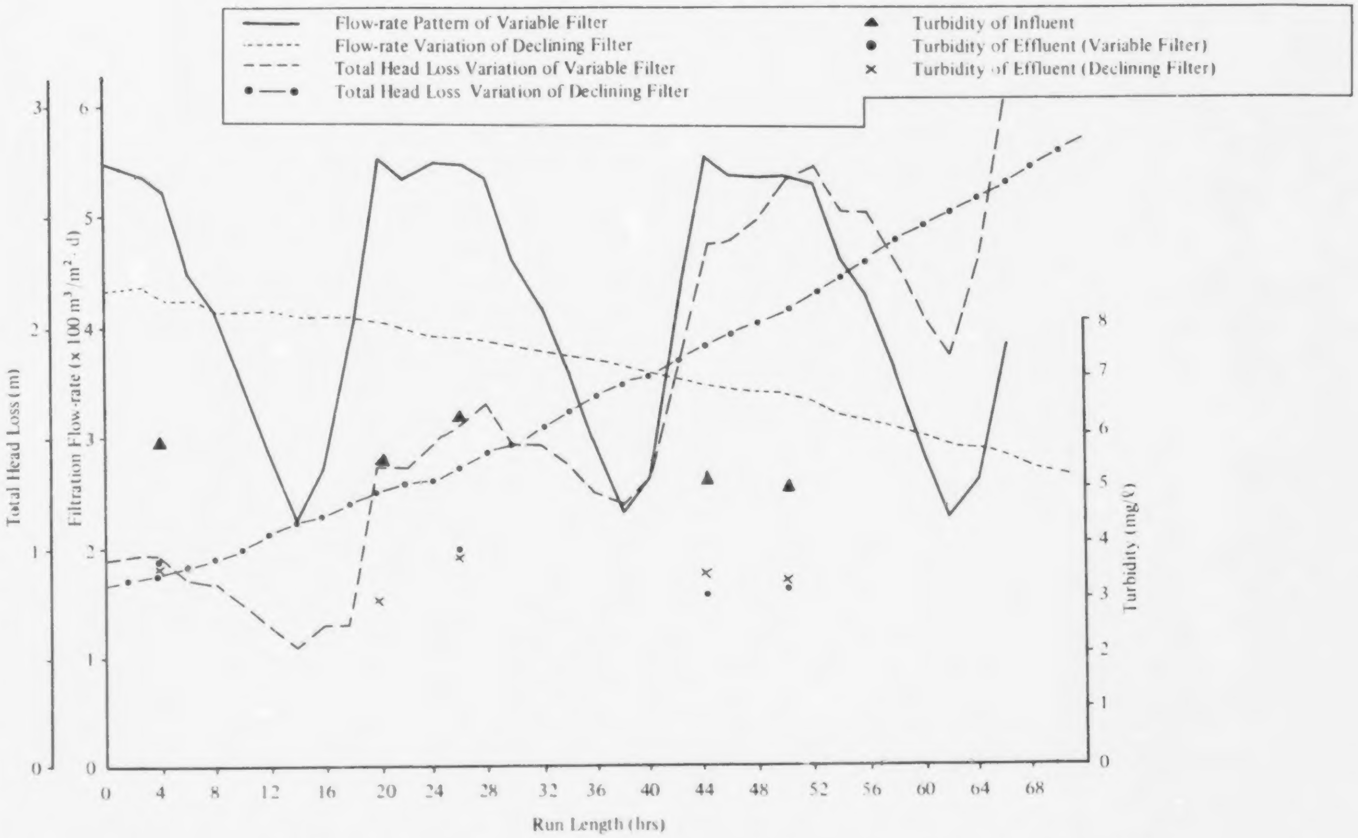


Fig. 3.10 Comparison of Variable Flow-rate and Declining Flow-rate on Flow-rate, Total Head Loss and Turbidity Variations in Down-flow Dual Media Filters

[Average Flow-rate of Variable Filter: Approx. 420 $m^3/m^2 \cdot d$
Initial Flow-rate of Declining Filter: 420 $m^3/m^2 \cdot d$]

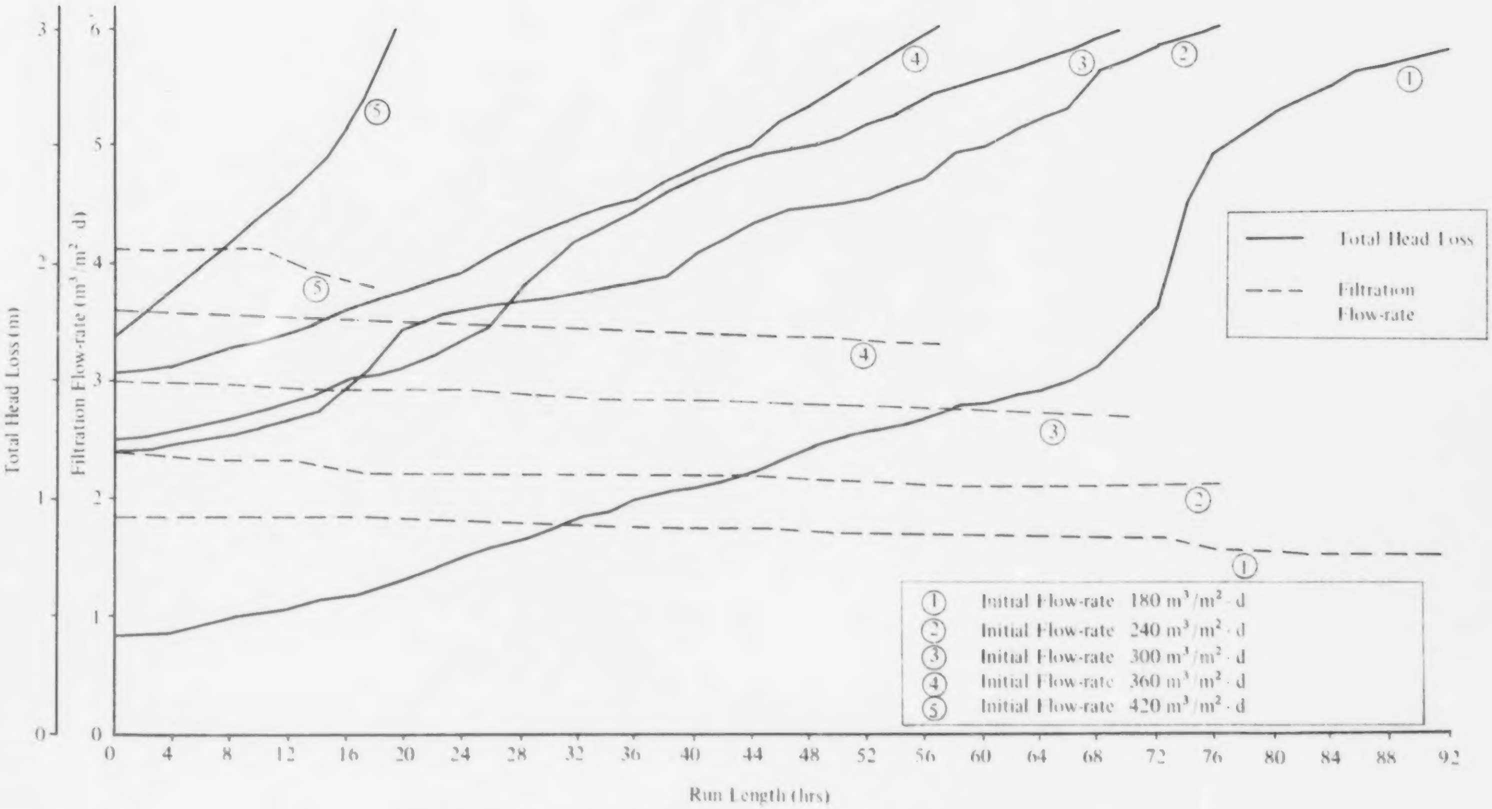


Fig. 3.11 Examples of Normal Operations of Up-flow Filters

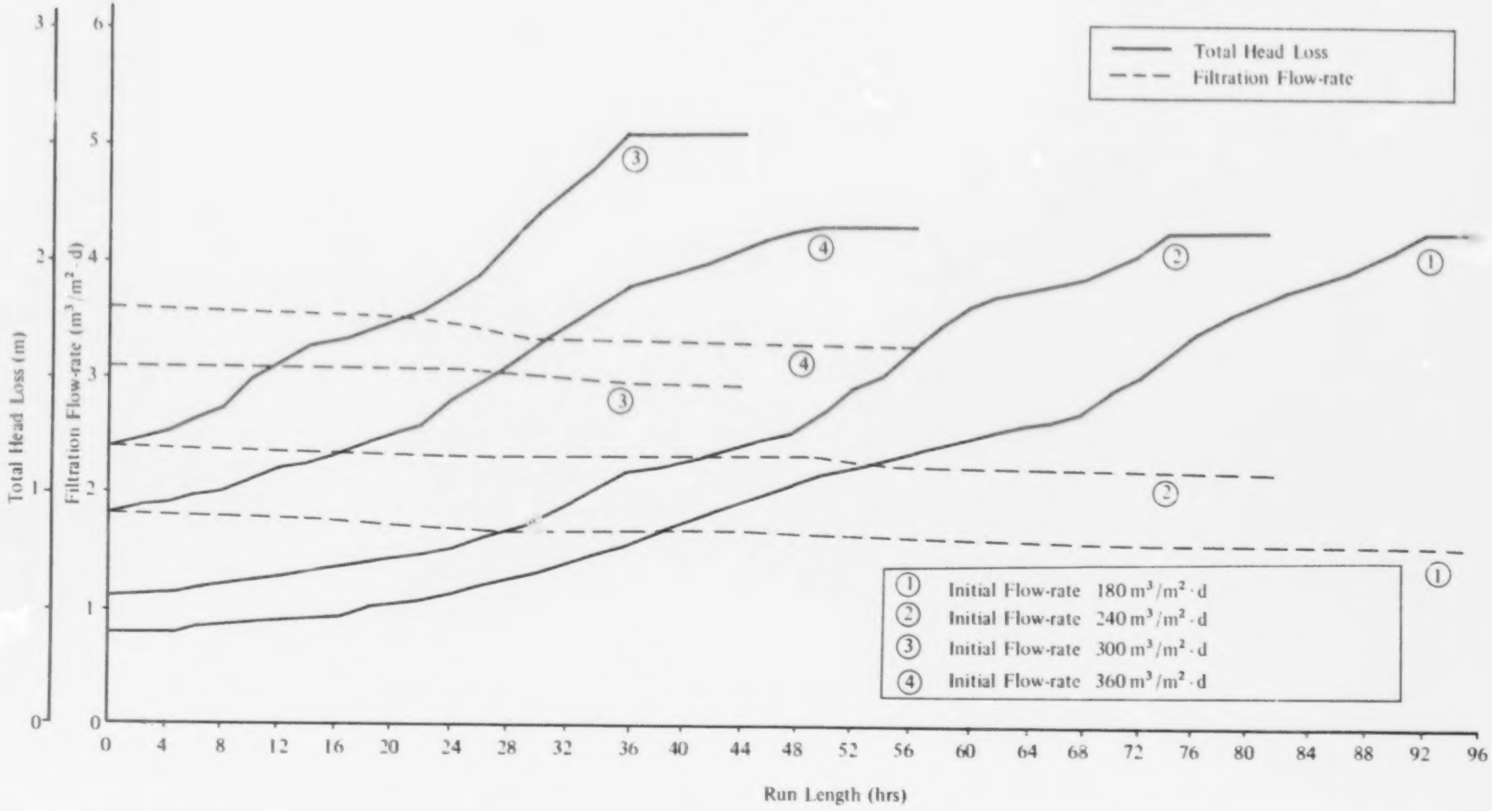


Fig. 3.12 Examples of Suspended Solids Leakage in Sand Beds at Up-flow Filter Test Operations

CHAPTER 4. METAL SALTS PRECIPITATION

4.1 Estimate of Concentration of Dissolved Phosphorus in Effluent and Removal of Dissolved Phosphorus 157
  4.1.1 Form of Phosphorus Used for Determination of Mole Ratio 157
  4.1.2 Relationship between Mole Ratio, Dissolved Phosphorus in Effluent, and Residual Dissolved Phosphorus 157
  4.1.3 Summary 160

4.2 Comparison of Coagulant Constant Feed and Mole Ratio Control 161
  4.2.1 Where Influent Flow is Set Constant 161
  4.2.2 Where Influent Flow is Changeable 162
  4.2.3 Considerations 164

4.3 Results of Pilot Plant Examinations 166
  4.3.1 Results of Wastewater Treatment 166
  4.3.2 Results of Sludge Dewatering 167

4.4 Development of High-Rate Flocculation-Sedimentation System – An Example of Advanced Waste Treatment Technical Development at a Private Company – 169
  4.4.1 Principles and Features of the Installation 169
  4.4.2 Pilot Plant Experiment at the Senda Sewage Treatment Plant, Hiroshima City 170
  4.4.3 Pilot Plant Experiment at the Shinhama Sewage Treatment Plant, Fukuyama City 171

4. METAL SALTS PRECIPITATION

4.1 ESTIMATE OF CONCENTRATION OF DISSOLVED PHOSPHORUS IN EFFLUENT AND REMOVAL OF DISSOLVED PHOSPHORUS

The Joint Working Group on Advanced Wastewater Treatment Technology conducted jar tests in the winter of 1974 on the phosphorus removal by metal salts precipitation. The coagulants used were alum ($Al_2(SO_4)_3 \cdot 18H_2O$), ferric chloride ($FeCl_3$, 26.2% Fe solution), and MICS (a mixture of alum and ferric sulfate, solution of Al_2O_3 (5.42%) and Fe_2O_3 (2.0%) available on market here in Japan.

The samples were influent to the primary settler and the secondary effluent, taken around 9:00 a.m. from 14 to 15 typical sewage treatment plants in Japan.

4.1.1 FORM OF PHOSPHORUS USED FOR DETERMINATION OF MOLE RATIO

In the test the mole ratios of metal salts to be dosed were set at 0.5, 1.0, 1.5, 2.0, 2.5 and 3.0 with the phosphorus (P: total phosphorous by persulfate digestion) in influent as a basis (e.g., Al/P, Fe/P, (Al +Fe)/P). No particular pH control was carried out.

Fig. 4.1 shows the relationship between phosphorus in influent and residual dissolved phosphorus (P-D, filtered through 1.2 μ millipore, persulfate digestion) for mole ratios of 1.0, 2.0, and 3.0 based on phosphorus in influent. Fig. 4.2 shows the relationship between the dissolved phosphorus in influent and residual dissolved phosphorus for mole ratios of 1.0, 2.0, and 3.0 based on dissolved phosphorus in influent. Fig. 4.2 provides better correlation than Fig. 4.1 when compard on the same mole ratio.

It is hence inferred that insoluble phosphorus in influent does not react upon Al dosed.

Figs 4.1 and 4.2 show only the results of alum treatment, but the results of treatments with ferric chloride and MICS also showed the same tendency.

For this reason, the discussions hereunder use the mole ratios based on the dissolved phosphorus in influent.

4.1.2 RELATIONSHIP BETWEEN MOLE RATIO, DISSOLVED PHOSPHORUS IN INFLUENT, AND RESIDUAL DISSOLVED PHOSPHORUS

As shown in Fig. 4.2, the concentration of dissolved phosphorus in influent and residual dissolved phosphorus are expressed by a straight regression line when the mole ratio obtained based on dissolved phosphorus concentration of influent is fixed at a constant value.

It is assumed that the relationship between the concentration of influent dissolved phosphorus ($P-D_I$) and the concentration of effluent dissolved phosphorus ($P-D_E$) is given by the following formula.

$$(P-D_E) = a \times (P-D_I) + b \quad \ldots\ldots (1)$$

Mole ratio: constant

$(P-D_E)$, $(P-D_I)$: in mg.P/lit.

a, b: constants

Then regression line is determined for each mole ratio, and a, b, and standard deviation of $(P-D_E)$ is calculated as shown in Table 4.1.

While the calculation for influent to the primary settler is made up to a mole ratio of 4, the calculation for the secondary effluent is up to a mole ratio of 3. This is due to the following reason.

In this test, with the concentration of phosphorus in the influent as a basis for determination of mole ratio, the mole ratios were increased up to 3 at an interval of 0.5. But it was found that the mole ratio can better be determined based on the concentration of dissolved phosphorus as stated in 1–2, and the results of the test were revised with the mole ratios determined according to the concentration of dissolved phosphorus.

On the other hand, the ratio of insoluble phosphorus in influent to phosphorus was larger in the influent to the primary settler; almost all results showed that when the mole ratio was 3.0 as calculated based on phosphorus in influent to the primary settler, the mole ratio calculated on the bais of dissolved phosphorus became more than 4.0.

In the case of secondary effluent, the ratio of insoluble phosphorus was comparatively small, so there was rarely the case that the mole ratio based on dissolved phosphorus was beyond 3.5. This is also evidenced by the fact that the secondary effluent show better correlations in Fig. 4.1.

The range of the concentration of dissolved phosphorus in the inlfuent used in the experiment was almost the same for ferric chloride, MICS and alum. Thus the range of the concentration of dissolved phosphorus in influent in which the computation results shown in Talbe 4.1 can be applied seems to be preferably 1.5 to 10 mg.P/lit. for influent to the primary settler and 0.5 to 3 mg.P/lit. for the secondary effluent. From Eq. (1), the removal of dissolved phosphorus (R_{P-D}) for the same mole ratio can be expressed as follows.

$$
\begin{aligned}
(R_{P-D})\ (\%) &= \left(1 - \frac{(P-D_E)}{(P-D_I)}\right) \times 100 \\
&= \left(1 - a - \frac{b}{(P-D_I)}\right) \times 100 \quad \cdots\cdots\cdots\cdots\cdots\cdots (2)
\end{aligned}
$$

Eq. (2) suggests that the mole ratio calculated from dissolved phosphorus in influent does not univocally determine the removal (R_{R-D}), but that in the same mole ratio the removal increases with increase in the concentration of dissolved phosphorus in influent as b is larger than O (see Table 1).

By way of example, Fig. 4.3 shows the relationship between the removal of dissolved phosphorus and the concentration of dissolved phosphorus with respect to the case where the influent to the primary settler is treated with alum.

In order to compare the performance of three coagulants, the concentrations of dissolved phosphorus in the effluent of influent to the primary settler treated with each coagulant were determined with a, and b in Table 4.1. The values used for the calculation are as follows. The mole ratios are 0.5, 1, 1.5, 2, 2.5, 3, 3.5, and 4.0, and the concentration of dissolved phosphorus in influent to the primary settler are 2, 4, 6, and 8 mg.P/lit. The results were as follows.

The concentrations of dissolved phosphorus after treatment with ferric chloride and MICS were respectively 1.61 times (0.94 to 9.41) and 1.05 times

(0.57~3.85) on the average as large as that after treatment with alum; namely, ferric chloride was inferior in removal of phosphorus to the other two.

One of the causes conceivable is pH after addition of coagulant. As already touched upon, the experiment under discussion was conducted without pH adjustment. As a consequence, as against the average pH value of raw sewage being 7.4 (6.6~8.1), the pH value after treatment with a mole ratio of 3, for example, was 6.8 on the average for all three coagulants – alum 6.8 (6.3~7.3), ferric chloride 6.8 (6.5~7.2) and MICS 6.8 (6.3~7.4). It is said that the optimum pH value for removal of phosphorus is 5.5 to 6.5 for alum and 4.5 to 5.0 for ferric chloride*.

The deviation of the average pH value, 6.8, from the optimum pH value after treatment at a mole ratio of 3 was 1.8 to 2.3 for ferric chloride and 0.3 to 1.3 for alum, and was greater in ferric chloride treatment than in alum treatment. It is therefore considered that the treatment with alum will lower residual dissolved phosphorus concentration more than with ferric chloride. The same tendency was observed with the secondary effluent.

With the values in Table 4.1 taken as a reference, the relationship between the more ratio and dissolved phosphorus concentration obtained after treatment of raw sewage with alum is shown in Fig. 4.4. According to Fig. 4.4 , the difference of the dissolved phosphorus concentration after treatment due to difference in the dissolved phosphorus concentrattion in influent decreases with increase in mole ratio, attains zero at a certain mole ratio (2.5 in this case), and then turns reversely.

The mole ratio at which the dissolved phosphorus in the effluent was held constnat irrespective of the concentration of dissolved phosphorus in the influent to the primary settler was 2.5 for alum, 3.4 for ferric chloride and 2.9 for MICS.

The same tendency was observed with the secondary effluent.

The removability of dissolved phosphorus in raw sewage and secondary effluent is compared in the following way. Dissolved phosphorus concentration in influent was taken as 2 and 3 mg./lit. which lie in the range in which Table 4.1 can be applied common to influent to the primary settler and secondary effluent, and as 0.5, 1, 1.5, 2, 2.5 and 3 in terms of mole ratio, and the corresponding residual total dissolved phosphorus concentrations were calculated. Then the calculation showed that the ratios of dissolved phosphorus in the secondary effluent to that in the raw sewage treated with alum, ferric chloride and MICS were 1.25 (1.06~1.50), 1.01 (0.78~1.16) and 1.06 (0.94~1.21) on the average respectively. From this, it is inferred that the secondary effluent is a little easier to remove phosphorus as compared with raw sewage, a little though the difference may be.

The relationship between a and b in Table 1 and mole ratios is shown in Fig.s 4.5 and 4.6 with respect to the treatment of raw sewage with alum.

In other conditions, reltionships between a and b in Table 4.1 and mole ratios are similar to Fig. 4.5, and 4.6.

If a and b can be expressed by the following formula (Eq. 3) according to

---

* U.S.E.P.A. Process Design Manual for Phosphorus Removal, Oct. 1971.

Figs. 4.5 and 4.6, then the substitution of Eq. (3) for Eqs. (1) and (2) will establish functions in terms of the mole ratio to phosphorus of dosed metal salts obtained based on the dissolved phosphorus concentration in influent as also of dissolved phosphorus concentration in influent, which can show residual dissolved phosphorus concentration and removal of dissolved phosphorus in the metal salts precipitation.

$$a = f(MR),\ b = g(MR) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

MR is mole ratio based on dissolved phosphorus in influent.

This time, however, we are not yet to refine the curves in Figs. 5 and 6 into such a function, and we will continue further study in this respect.

Tables 4.2, and 4.3 show the relationships between the dosages of alum, ferric chloride, and MICS and mole ratios necessary for attaining the target dissolved phosphorus concentrations in effluent of 0.1, 0.2, 0.3, 0.5, and 1.0 mg.P/l, and the target removal of dissolved phosphorus of 98, 95, 90, 85, and 80%, and dissolved phosphorus in influent.

4.1.3 SUMMARY

i) In case wastewater is treated with metal salts, like aluminum salts, and iron salts, to remove phosphorus, these metal salts react upon dissolved phosphorus, but not upon insoluble phosphorus.

ii) When the mole ratio, Al/, Fe/P, or (Al + Fe)/P, calculated based on dissolved phosphorus concentration in influent is set constant, there is a rectilinear relationship between the dissolved phosphorus concentration in influent and that in the effluent.

iii) The concentration of dissolved phosphorus in influent to which the coefficient shown in Table 4.1 prepared for determing the concentration of dissolved phosphorus in effluent and mole ratio based on the concentration of dissolved phosphorus in influent can be applied should preferably be in the range of 1.5 to 10 mg.P/lit. for influent to the primary settler and 0.5 to 3 mg.P/lit. for secondary effluent.

iv) Even if the mole ratio is set constant, the removal of dissolved phosphorus increases with increase in the concentration of dissolved phosphorus in influent.

v) The difference of the concentration of dissolved phosphorus after treatment due to difference in the concentration of dissolved phosphorus in influent decreases with increase in mole ratio, is reduced to zero at a certain mole ratio, and then turns up from there on.

vi) So far as the removability of phosphorus is concerned, alum and MICS lie almost on the same level, while ferric chloride is poorer than them. In the experiment using these three metal salts, pH control was not carried out.

vii) Removal of dissolved phosphorus is a little better in secondary effluent than in raw sewage, though the difference is not so large.

viii) The results obtained in the experiment concerning the relationship betweeen target removal of dissolved phosphorus and coagulant dosage are shown in Tables 4.2 and 4.3.

4.2 COMPARISON OF COAGULANT CONSTANT FEED AND MOLE RATIO CONTROL

It is clarified under item 4.1 that in the phosphorus removal by metal salts precipitation, the dissolved phosphorus concentration in effluent is dependent on the mole ratio of dosed metal salts to dissolved phosphorus in influent as well as on the concentration of dissolved phosphorus in influent.

In sewage treatment plant, usually, the influent flow and the dissolved phosphorus concentration in influent are changeable.

In order to minimize the loading of dissolved phosphorus in effluent mole ratio control method in which the metal salts dosage is controlled to make the mole ratio constant and constant feed method in which metal salts are fed at a constant rate were put to comparison study with reference to the same coagulant and the same total dosage.

To this purpose, experiments were carried out in the following way.

"A" sewage treatment plant in the suburbs of Tokyo covering a population of 11,300 with a separate sewer system and undertaking secondary treatment in step aeration process with two hours of aeration time, was picked up.

An automatic sampler was used to take its secondary effluent every hour for 24 hours in order to obtain influent for the experiment, and 24 influents were analyzed of dissolved phosphorus.

In the constant feed method, alum was dosed to make Al/P mole ratios of 1, 2 and 3 with respect to the mean value of the concentration of dissolved phosphorus in influents. In the mole ratio control method, alum was dosed so that Al/P mole ratio could be 1, 2 and 3 with respect to dissolved phosphorus in each influent. The samples added with alum (24 x 2 x3 = 144 test tubes) were put to jar test, and total dissolved phosphorus (H_2SO_4 digestion) was analyzed each.

4.2.1 WHERE INFLUENT FLOW IS SET CONSTANT

Fig. 4.7 shows time-dependnet changes of inlfuent flow rate and the concentration of dissolved phosphorus in both influent and effluent.

Fig. 4.8 refers to constant feed method and mole ratio control method, and shows their relationships between the concentration of dissolved phsophorus in influent and the removal of dissolved phosphorus. The concentration of dissolved phosphorus in influent recorded a maximum value of 6.73 mg.P/lit. at noon, a minimum 1.73 mg.P/lit. at 7 a.m. and 3.36 mg.P/lit. on the average. The ratio of the maximum to the minimum was 3.9. In the constant feed method, the maximum to minimum ratio of dissolved phosphorus in effluent was 7.5, 14.9 and 17.5 for the mole ratios of 1, 2, and 3, respectively, while in the mole ratio control method, the corresponding maximum to minimum ratio was 3.1, 2.8 and 4.4. At any mole ratio, the mole ratio control method could make smaller the change in the concentration of dissolved phosphorus in effluent than the constant feed method.

From Fig. 4.8, it is found that in the constant feed method, the removal of dissolved phosphorus decreased with increase in the concentration of dissolved phosphorus in influent.

This is because as alum dosage was constant, the mole ratio (Al/P) decreased with increse in the dissolved phosphorus concentration in influent. For this reason, the lower the dissolved phosphorus concentration in influent, the smaller the residual total phosphorus concentration, sending up the maximum to minimum ratio of dissolved phosphorus in effluent. On the other hand, the mole ratio control method increased the removal of dissolved phosphorus with increase in the concentration of total dissolved phosphorus in influent just as the tendency discussed under item 4.1; namely, the maximum to minimum ratio of dissolved phosphorus in effluent did not become so large.

Assuming that the influent flows were equal to the average flow rate of "A" sewage plant at that sampling day (87 m^3/hr), the dissolved phosphorus loadings of influent and effluent, average removal and concentration are as shown in Table 4.4.

According to Table 4.4, the mole ratio control can reduce dissolved phosphorus in the effluent irrespective of mole ratios as compared with the constant feed method. At mole ratios of 1, 2, and 3, the dissolved phosphorus loading in the effluent in the mole ratio control method was 91%, 86% and 81% of the corresponding values achievable by the constant feed method.

Fig. 4.9 shows the relationship between the dissolved phosphorus concentration in influent and the difference in dissolved phosphorus concentration after treatment between in the constant feed method and in the mole ratio control method. According to Fig. 4.9, it is found that a turn of tendency is seen at an average dissolved phosphorus concentration of 3.36 mg.P/lit.

Assuming that these two demarcated ranges can each be replaced with a straight line, respective regression straight lines can be determined as shown in Table 4.5.

It is found from Table 4.5 that irrespective of whether the mole ratio is 1, 2 or 3, the slope (a) is larger in the case where the average dissolved phosphorus concentration in influent is larger than 3.36 mg.P/lit. as compared with the case where the average concentration is lower.

This difference in slopes means that the constant feed control increases the solved phosphorus in the constant feed method decreases with increase in the concentration of dissolved phosphorus in influent whereas the mole ratio control increases it to the contrary.

This difference is slope means that the constant feed control increases the dissolved phosphorus loading in the effluent than the mole ratio control method.

4.2.2 WHERE INFLUENT FLOW IS CHANGEABLE

The constant feed in the case of influent flow change assumed a method of keeping the coagulant dose per unit volume of influent at a constant value.

In the mole ratio control method, dosage was so made as to be proportional to the influent flow rate and the concentration of dissolved phosphorus in the influent in an attempt to keep the mole ratio (Al/P) always constant.

Fig. 4.10 shows time-dependent changes of dissolved phosphorus loading in the influent and effluent which are determined by multiplying the influent flow rate shown in Fig. 4.7 by the concentrations in fluent and effluent of dissolved phosphorus.

In connection with this, daily total laoding, removal, average concentration (quotient obtained by dividing daily dissolved phosphorus loading in the effluent by daily influent flow), and Al consumption are shown in Table 4.6.

According to Table 4.6, it is found that dissolved phosphorus loading is lower in the mole ratio control method than in constant feed method.

However, the mole ratio control method consumes 6.7% more Al than the constant feed method. This is because the influent flow was comparatively large when the concentration of dissolved phosphorus in influent was higher than the average.

In order to make comparison on a same Al consumption basis, the mole ratios for the mole ratio control were taken as 1/1.067 = 0.94 and in the same way, 1.87 and 2.81, and the corresponding loadings of dissolved phosphorus in the effluent were determined from Fig. 4.11 and compared with the values listed in Table 4.3 for the constant feed method.

In this case, at mole ratios of 1, 2 and 3 in the constant feed method, the mole ratio control method reduced daily total loading of dissolved phosphorus in effluent to 93%, 91% and 88%, respectively.

Qualitatively speaking, there may be the following effects of the way of correspondence between the time change of influent flow and the time change of dissolved phosphorus in influent on the daily total loading of dissolved phosphorus in the effluent.

i) Where the influent flow and the dissolved phosphorus concentration in influent are proportional to each other:

   When the constant feed method and mole ratio control method are practised on the same mole ratio, the mole ratio control method even more reduces the dissolved phosphorus loading in the effluent, because when the concentration of dissolved phosphorus in influent is larger than the average in which the mole ratio control method better removes dissolved phosphorus than the constant feed method, the flow rate is larger than the average.

   Al dosage, however, becomes larger in the mole ratio control method owing to the reason explained above.

   Accordingly, if the mole ratio control is carried out on the same Al consumption with the constant feed method, the mole ratio of the mole ratio control becomes smaller than referred to in the constant feed mehod.

   The mole ratio control method is agreeable in that the dissolved phsophorus loading of influent at a point where the dissolved phosphorus removal becomes larger than in the constant feed method, is larger, but is disagreed in that the mole ratio becomes smaller.

ii) Where the influent, flow and the dissolved phosphorus concentration in

influent are in inverse proportion to each other:
This is quite contrary to the tendency examined under item i) above. Namely, the mole ratio control is disagreed in that the dissolved phosphorus loading in the influent at a point where the removal becomes larger than the constant feed mehtod becomes smaller, but is agreeable in that the mole ratio becomes larger.

To follow Case (a), 9:00 flow which was maximum and 13:00 dissolved phosphorus concentration which was also maximum were adjusted to have concurred, and calculations were made.

To follow Case (b), 2:00 flow which was minimum and 13:00 concentration which was maximum were adjusted to have concurred, and calculations were made. Fig. 4.12 shows the relationship between the flow and concentration of dissolved phosphorus for respective cases.

The computation results appear in Table 4.7. Table 4.7 also shows a case of constant flow and another in which flow and phosphorus concentration concurred really.

From Table 4.7, the comparison between the constant feed and the mole ratio control in which the coagulant dosages are the same in Al reveals that which method is more advantageous than the other is dependent on (1) mole ratio and (2) way of correspondence between time changes of influent flow and the dissolved phosphorus concentration in influent.

In case where the mole ratio is 1 as referred to in the constant feed, the way of correspondence between flow and concentration makes little difference, and dissolved phosphorus loading after treatment in mole ratio control is 91 to 94% of that after constant feed treatment. Similarly, the mole ratio of 2 develops a loading range of 86 to 91%.

At the mole ratios of 3 in the constant feed, the loading changes depending on the mode of correspondence between the flow and concentration; when the flow tends to become in reciprocal proportion to phosphorus concentration, the mole ratio control method becomes more advantageous.

4.2.3 CONSIDERATIONS

It is found that the mole ratio control method can reduce dissolved phosphorus loading in the effluent more than the constant feed method even when compared on the same basis of alum consumption. (See Table 4.7)

It should be noted however that this take "A" sewage treatment plant as a model and that the method presupposes that the measurment of flow and dissolved phosphorus concentration in influent is made continuously without time delay.

"A" sewage treatment plant is a separate sewer type sewage treatment plant located in a Danchi (housing complex) and is considered to have very changeable concentration of dissolved phosphorus in the raw sewage, and secondary effluent. For example, the Toba Sewage Treatment Plant in Kyoto (influent flow: approx. 4.20 million $m^3/d.$) was found on a survey to have the phosphorus concentration in the secondary effluent changed in 24 hrs from a maximum of 2.12 mg.P/lit. to a minimum of 1.76 mg.P/lit., with the maximum to minimum ratio at 1.20. If the metal salts precipitation is applied to this less changeable sewage for removal of phosphorus, the mole ratio control will not be able to exhibit the effects shown in this experiment.

The measurement of influent flow will be accomplished easily in real time if a suitable instrument is applied.

Even with the colorimetric analysis method which is the most promising at present, the measurement of the concentration of dissolved phosphorus in the influent will unavoidably take a time lag of 20 to 60 min.

The applicability of the mole ratio control method should therefore be judged after due consideration of not only the results of jar test, but also considerations time-dependent pattern of influent dissolved phosphorus concentration and influent flow, as well as response characteristics of phosphorus analyzer available.

4.3 RESULTS OF PILOT PLANT EXPERIMENTS

The Yokosuka Advanced Wastewater Treatment Pilot Plant has two systems of precipitation; one using lime and the other using metal salts. Dealt with here are the results of experiments on the precipitation using alum as coagulant in the metal salts system and on the filtration. The operating conditions of alum precipitation and filtration are shown in Table 4.8. Both flow rate of the influent, which is secondary effluent, and alum dosage were kept constant throughout the experiments.

4.3.1 RESULTS OF WASTEWATER TREATMENT

Table 4.9 shows the average results of wastewater treatment.

a. Removal of suspended solids

Turbidity and suspended solids were removed about 30% by precipitation and about 90% after filtration. With a view to improving the removal of suspended solids in the sedimentation tank, the effects of dosage of ferric chloride and anionic polymer as flocculation aids and of returning sludge were examined.

The mean values of the results are shown in Table 4.10, According to Table 4.10, the concentration of suspended solids in the effluent of the sedimentation tank is found unaffected by a dose of 0 to 5.0 mg.Fe/l of ferric chloride, a sludge return ratio to influent of 0 to 30% and a dose of 0 to 0.5 mg/l of polymer.

The concentration of suspended solids was relatively constant within the range of 10 to 15 mg/l, independent on these test conditions. This will be clarified by further investigations scheduled, including a test in which the overflow rate is to be changed in the range of 10 to 100 m^3/m^2 d and a test of a shallow settling device.

The Yokosuka pilot Plant is equipped with two filters, different in constitution of media. (See Table 4.11). The media of the No.2 filter is coarser than that of No.1 filter.

The quality of the filter effluent shown in Table 4.9 was obtained by analysis of a composite sample consisted of effuents from No.1 and No.2 filters.

The filtration run length at a total head loss of 2 m was 15.0 hrs and 18.5 hrs on the average for No. 1 filter and No.2 filter, respectively. Namely, the filtration run length of No.2 filter was 1.32 times as long as that of No. 1 filter. On the other hand, the average concentration of suspended solids in filter effluent was 1.0 mg/l and 1.6 mg/l for No. 1 filter and No. 2 filter, respectively, demonstrating that No. 1 filter, which used fine media, excelled No.2 filter.

b. Removal of phosphorus

Phosphorus and orthophosphate were removed respectively 90.3% and 91.7% after filtration. Their average concentrations in the filter effluent were 0.173 mg.p/l and 0.109 mg.P/l, respectively as shown in Table 4.9. The dissolved phosphorus in effluent treated by metal salts can be estimated by the method discribed in Section 4.1 if the mole ratio of the dosed metal salts to phosphorus based on the dissolved phosphorus concentration in influent and the dissolved phosphorus in influent are known.

In the period when the data shown in Table 4.9 were collected, the dissolved phosphorus was not analyzed. Therefore, the comparison of estimated and measured concentrations in the effluent from sedimentation fank will be made using data shown in Table 4.10 and the values of coefficients listed in Table 4.1.

The average concentration of the dissolved phosphorus in influent	1.411 mg.P/l
Mole ratio (Al/P)	3.19
Coeffecient a at mole ratio 3.19	0
Coeffecient b at mole ratio 3.19	0.25

Concentration of dissolved phosphorus in effluent (mg.P/l) = a·(concentration of dissolved phosphorus in influent) + b = 0 × 3.19 + 0.25 = 0.25 (mg.P/l)

One the other hand, the average concentration of dissolved phosphorus in effluent shown in Table 4.10 is 0.130 or roughly 50% of 0.25 mg.P/l of estimated value. The results of experiments by the Joint Working Group on Advanced Wastewater Treatmnet discussed under Section 4.1 include data obtained from the Shitamachi Sewage Treatment Plant, Yokosuka, where the pilot plant under discussion is installed. Here, a similar result was obtained. Namely, when secondary effluent is treated by alum precipitation at mole ratio of 3, dissolved phosphorus in the effluent is estimated to be 0.279 mg.P/l, while the actual concentration was about half the estimate or 0.142 mg.P/l (concentration of dissolved phosphorus in influent: 0.686 mg.P/l). Thus it is inferred that the dissolved phosphorus in the secondary effluent of the Shitamachi Sewage Treatment Plant is easy to remove by alum addition compared with effluents of other sewage treatment plants, and the causes are to be studied.

In the experimental period during which the data listed in Table 4.9 were collected, the average concentration of dissolved phosphorus in the in fluent of the filter was 0.124 mg.P/l. On the other hand, phosphorus concentration in effluents from No.1 filter and No.2 filter were 0.075 mg.P/l and 0.091 mg.P/l, respectively. Hence in No.1 filter, at least 0.049 mg.P/l or 40% of dissolved phosphorus was removed, while in No.2 filter 0.033 mg.P/l or 27% was removed.

c. Removal of organic matter and bacteria

As to removal of organic motter by alum precipitation and filtration, removal of BOD was the highest among BOD, COD by potassium permanganate, and COD by potassium dichromate Removals of COD by potassium dichromate and COD by potassium permanganate were almost on the same level. The value of COD by potassium permanganate was about one third that of COD by potassium dichromate.

The common bacteria were removed only 38.7% before filtration, but coli-form bacteria were removed as much as 90.6%, showing that coli-form bacteria are easier to remove than common bacteria.

4.3.2 RESULTS OF SLUDGE DEWATERING

Alum sludge from an alum sedimentation tank was thickened and dewatered by means of a centrifuge. The centrifuge used for the test was the smallest one available on market for use in sewage sludge dewatering, having a standard sludge feed rate of 1 m^3/hr., and was modified for the test purposes.

The characteristics of feed sludge, test conditions and dewatering results are shown in Table 4.12. According to Table 4.12, it is found that alum sludge is hard to dewater with a centrifuge without chemical dosage.

Even with the result of test No.4 which was the best among the results, the rate of total solids captured in dewatered sludge was 48.4%, and water content of dewatered sludge was 88.2%. At that time, the suspended solids in the centrate was 7,000 mg/1.

Then, alternative dewatering methods are called for, which may include: (1) centrifugal dewatering with desage of polymer, (2) dewatering after being mixed with organic sludge, (3) dewatering after freeze conditioning, and (4) use of filter press or vacuum fieter.

Preliminary study of freeze conditioning was carried out using a home freezer to freeze the alum sludge. The settling test for the alum sluage showed that the ratio of the sludge volume after 30 min. settling to the intial volume was 27% for the freeze conditioned sludge as against 95% for the non-treated, evincing that the thickening rate can be improved sharply by the freezing and melting process. This held also true with the lime sludge.

4.4 DEVELOPMENT OF HIGH-RATE FLOCCULATION-SEDIMENTATION SYSTEM – AN EXAMPLE OF ADVANCED WASTE TREATMENT TECHNICAL DEVELOPMENT AT A PRIVATE COMPANY –

In Japan, several types of chemical flocculation-sedimentation installation have been introduced at water purification plants, etc. The chemical flocculation-sedimentation system described here has been developed by a private company based on a new concept. This system has some advantages compared with conventional ones: for example, it can provide larger overflow rate, and floc becomes in a pellet state in the sedimentation installation thereby, facilitating its treatment.

Because this new installation can remove the residual organic substances and ortho-Phosphate in the secondary effluent with a high efficiency within a very small site, many sewage works engineers in several cities have paid attention to it with interest.

This is a report on pilot plant studies in Hiroshima and Fukuyama Cities.

This system will be assessed as an actual facility by the Japan Sewage Works Agency's Committee of Technology Evaluations in 1976.

4.4.1 PRINCIPLES AND FEATURES OF THE INSTALLATION

This system is a flocculation-sedimentation installation of the sludge blanket type in which both polymers and metal salts coagulants such as aluminum sulphate and ferric chloride, are used. Since flocs forming the sludge blanket are huge pellets, as big as 5 to 10 mm, the installation has a good characteristic in terms of settling, has fewer carry-overs of flocs and can be designed to allow overflow rate of 300 m/d or more.

The mechanism of formation of such huge pellets has not been fully understood. But several experiments have proved that this installation is very effective to remove colloidal organic matter and phosphate.

The pilot scale experimental facility used for the study is shown in Fig. 4.13. Its major portion is composed of a 1.8m × 1.8m square flocculation-sedimentation tank, a mixing tank which mix water and coagulants and tanks to dissolve coagulants.

An electromagnetic flow meter and an inflow control valve are provided. A predetermined quantity of inflowing water is fed to the mixing tank and mixed with inorganic coagulant (aluminum sulphate). Polymer is fed with pressure to the piping between the mixing tank and the installation (flocculation-sedimentation tank). The influent is sent to the installation from its bottom. Water and flocs are separated when they passethrough the sludge blanket zone consisting of granular flocs. Then water is discharged from the overflow weir. The separated flocs gradually grow by the coagulation action given by the inside impeller anto granular flocs which consists of the sludge blanket. The level of the sludge blanket which goes up gradually is detected by the ultrasonic sludge level sensor mounted a predetermined hight. And by the operation of the sludge drawing valve which is connected electrically with the sludge level sensor, the sludge blanket zone is maintained at a fixed level.

Originally, the installation was designed to use a square tank. But later it was found that a square tank difficulties in maintaining the sludge blanket zone at a fixed state at corners. Therefore, the installation was remodeled to use a octagonal tank, in which the sludge blanket zone could be kept at a fixed state. In an actual facility, it is considered that circular type would be the best. In that case, the maximum diameter will be 5.5 m considering the limit for the peripheral velocity of impeller not to destroy the floc.

4.4.2 PILOT PLANT EXPERIMENT AT THE SENDA SEWAGE TREATMENT PLANT, HIROSHIMA CITY

Experiments as to use of this high-rate flocculation-sedimentation system as a tertiary treatment facility of waste water was conducted from Feburary to April, 1970, using the secondary effluent from the Senda Sewage Treatment Plant, Hiroshima City, The Plant is a medium-sized treatment plant having a planned serving population of 100,000 and adopting conventional activated sludge system.

i) Water quality of influent

- Water quality of the secondary effluent (that is, influent to the high-rate flocculation-sedimentation system) during the period of experiments was as shown in Table 4.13. Fig. 4.14 shows protted relations between suspended solids and total COD_{Mn}. in secondary effluent. From this figure, one can find it is known that even though suspended solids are completely removed, about 4 to 5 mg/l of soluble COD_{Mn} still remains.

ii) Time needed for accumulation of pellet-state sludge in the starting of operation

- Operation was started under the following conditions:
  Quantity of flow, 50m^3/hr; overflow rate, 250mm/min; dosing rate of alum, 50mg/l and dosing rate of polymer, 1.0mg/l. Under these conditions, the time needed for the blanket level to reach the predetermined height (i.e., 1.5m above the inflow port) was about 10 hours when treating low SS secondary effluent like one shown in Table 4.13.

iii) Effects of the dosing rate of alum on flocculation

- Jar tests: Laboratory tests have been conducted with the secondary effluent containing 15mg/l of suspended solids and 13.2mg/l of COD_{Mn} to which a various quantity of alum was added. A good flocculation was observed when alum dosage was 40mg/l as $Al_2(SO_4)_3 18H_2O$ and flocculation was excellent when dosage was 50mg/l.
- Continuous tests: Effects of alum dosing rate on the quality of effluent were examined under the conditions as follows: overflow rate, 250mm/min; retention time in buffled flocculator, 80 sec.; polymer dosing rate, 1mg/l; and dosing rate of alum, 50 to 60mg/l. Within the range of alum dosing rate examined, differences in performance were not so significant. But in order to form core flocs in the sludge blanket, the alum dosing rate must be 40 to 50mg/l. If the rate is 30mg/l, the size of core flocs is too small and the quality of effluent tends to be slightly worse.

iv) Optimum period between dosing of alum and dosing of polymer

- In this experiment, overflow rate was set at 250mm/min, dosing rate of alum, at 50mg/l, that of polymer, at 1mg/l, and mixing intensity in the flocculation tank at 0.62 (kg.f-m/m^3. sec). Under these conditions, the optimum period between dosing of alum and that of polymer was found to be 80 seconds or so.

v) Maximum overflow rate for stable operation

- The effect of overflow rate on the stability of the sludge blanket was studied under the condition of 50mg/l of alum dosing rate and 1mg/l of that of polymer As a result, it was found that the critical up flow velocity is in the vicinity of 300mm/min If the installation is designed to obtain more uniform distribution of velocity throughout the sludge blanket, overflow rate may be increased up to about 350mm/min.

vi) Thickening characteristic of excess flocculated-sedimented sludge

- Thickening tests were carried out using samples taken from blanket slurry which was formed under the conditions of 250mm/min of overflow rate, 40 to 50mg/l of alum dosing rate and 1.0mg/l of polymer dosing rate. A column of the diameter of 70mm and the height of 120mm was used for the tests. An example of the results is shown in Fig. 4.15 Sludge concentration reached 28 to 35g/l after 24 hours. From this result, it may be said that thickening characteristic is comparatively good.

vii) Quality of effluent

- Table 4.13 shows quality of effluent which was obtained at the optimum dosing rates of coagulants, and at overflow rates less than 280mm/min. It is considered that effluent quality has no relation directly with changes in the quality of influent. Insoluble COD_{Mn} and insoluble BOD were removed very effectively. Concentration of COD_{Mn} remaining in effluent is about 10% lower than that of soluble COD_{Mn} in influent. From this, it can be considered that part of soluble COD_{Mn} in influent was also removed. Orthophosphate was removed very effectively although the orthophosphate concentration was relatively low, 0.8~1.4mg/l.

4.4.3 PILOT PLANT EXPERIMENT AT THE SHINHAMA SEWAGE TREATMENT PLANT, FUKUYAMA CITY

Experiments at the Shinhama Sewage Treatment Plant have been carried out with effluent from the plant. Worse in quality than that of the Senda Sewage Treatment Plant. They have been conducted with the following three types of effluents:

Type A: Effluent from modified activated sludge treatment process which is overloaded.

Type B: Mixed effluent of three types of effluents, i.e., that from the conventional activated sludge treatment facility, that from the overloaded modified activated sludge treatment facility, and that from the primary setting tank.

Type C: Mixed effluent of two types of effluents, i.e., that from the overloaded modified activated sludge treatment facility, and that from the primary settling tank.

Qualities of these types of influents during each test period are shown in Table 4.14. Relations between COD_{Mn} and COD_{cr} of Type A and Type B are shown in Fig. 4.16. In the Figure both total COD and soluble COD are plotted and a high correlation is found in both cases.

As with experiments in Hiroshima, coagulants used in this pilot plant tests were alum and polymer. The dosing rates of them are shown in Table 4.14. The overflow rate was in the range of 330 to 350mm/min.

Quality of effluent obtained in this experiment is shown in Table 4.14. Also, Fig. 4.17 shows removal characteristics of COD_{Mn} in the experiment. Summary of the results are as follows:

a. Total COD_{Mn} of effluent depends on the concentration of soluble COD_{Mn} in influent.

b. Total COD_{Mn} in effluent is always lower than soluble COD_{Mn} in influent. From this, it is concluded that insoluble COD_{Mn} in influent is removed almost completely and a part of soluble COD_{Mn} in influent (15~20%) is also removed.

c. Relations between soluble COD_{Mn} in influent and that in effluent are shown in solid line in Fig. 4.18. Relations between soluble COD_{Mn} in influent and total COD_{Mn} in effluent are shown in a broken line in the same figure. From Fig. 4.18, it can be considered that 35% of soluble COD_{Mn} in influent was transferred into insoluble COD_{Mn} by the flocculation-sedimendation process, 30 to 35% of which was removed in the sludge blanket.

d. Fig. 4.19 shows the relation between total COD_{Mn} and soluble COD_{Mn} in effluent. From this Figure, it can be said that 80% of total COD_{Mn} in effluent is soluble.

Insoluble COD_{Mn} carried over from the overflow weir of this installation is floc with comparatively low specific gravity which is formed by conversion of soluble COD_{Mn} in influent into insoluble one through the flocculation-sedimentation process.

Table 4.1 Coefficients a and b, and Standard Deviation of Regression Line

Coagulant	Mole ratio	Influent to the primary settler			Secondary effluent		
		a	b	σ	a	b	σ
$Al_2(SO_4)_3$	0.5	0.743	0.149	0.218	0.692	0.156	0.122
	1	0.472	0.390	0.266	0.465	0.272	0.170
	1.5	0.223	0.632	0.255	0.266	0.316	0.211
	2	0.074	0.693	0.169	0.119	0.357	0.209
	2.5	0.000	0.610	0.158	0.057	0.318	0.188
	3	-0.029	0.497	0.144	0.011	0.271	0.133
	3.5	-0.028	0.368	0.121			
	4	-0.039	0.343	0.091			
$FeCl_3$	0.5	0.671	0.381	0.194	0.807	0.030	0.147
	1	0.448	0.486	0.315	0.545	0.207	0.153
	1.5	0.248	0.703	0.363	0.360	0.306	0.156
	2	0.088	0.877	0.402	0.234	0.282	0.104
	2.5	0.014	0.782	0.388	0.146	0.270	0.098
	3	0.006	0.539	0.278	0.094	0.240	0.071
	3.5	0.001	0.381	0.238			
	4	-0.010	0.332	0.042			
MICS	0.5	0.703	0.214	0.128	0.810	0.024	0.103
	1	0.457	0.368	0.240	0.561	0.163	0.099
	1.5	0.239	0.517	0.276	0.331	0.279	0.138
	2	0.115	0.490	0.229	0.148	0.358	0.131
	2.5	0.027	0.469	0.201	0.029	0.373	0.113
	3	-0.002	0.363	0.181	-0.002	0.328	0.087
	3.5	-0.007	0.230	0.103			
	4	-0.008	0.167	0.067			

The number of data is 14 or 15.

Table 4.2 Objective of Dissolved Phosphorus Removal vs. Coagulant Dosage and Mole Ratio

P-D in influent (mg/ℓ)	Objective P-D in effluent (mg/ℓ)	$Al_2(SO_4)_3$		$FeCl_3$		MICS		Objective P-D removal (%)	$Al_2(SO_4)_3$		$FeCl_3$		MICS	
		Mole ratio	Al(mg/ℓ)	Mole ratio	Fe(mg/ℓ)	Mole ratio	Al + Fe (mg/ℓ)		Mole ratio	Al(mg/ℓ)	Mole ratio	Fe(mg/ℓ)	Mole ratio	Al + Fe (mg/ℓ)
2	0.1	–		–		–		98	–		–		–	
4		–		–		–			–		–		–	
6		4.1	21.4	–		–			4.0	20.9	–		4.0	23.3
8		3.7	25.8	–		(4.1)	(31.8)		3.4	23.6	–		3.5	27.2
10		3.5	30.5	–		3.7	35.9		3.0	26.1	–		3.3	32.0
2	0.2	–		–		3.6	7.0	95	–		–		–	
4		3.8	13.2	–		3.5	13.6		3.8	13.2	–		3.6	14.0
6		3.5	18.3	–		3.5	20.4		3.0	15.7	3.8	41.0	3.2	18.6
8		3.2	22.3	–		3.4	26.4		2.8	19.5	3.5	50.4	2.9	22.5
10		3.0	26.1	–		3.3	32.0		2.6	22.6	3.3	59.4	2.8	27.2
2	0.3	3.6	6.3	(4.2)	(15.1)	3.3	6.4	90	–		–		3.6	7.0
4		3.3	11.5	3.9	28.1	3.2	12.4		3.0	10.4	3.5	25.2	2.9	11.3
6		3.0	15.7	3.9	42.1	3.2	18.6		2.5	13.1	2.9	31.3	2.6	15.1
8		2.9	20.2	3.8	54.7	3.2	24.9		2.3	16.0	2.6	37.4	2.4	[illegible]
10		2.8	24.4	3.7	66.6	3.1	30.1		2.2	19.2	2.5	45.0	2.3	22.3
2	0.5	2.8	4.9	3.1	11.1	2.5	4.9	85	3.6	6.3	(4.2)	(15.1)	3.3	6.4
4		2.7	9.4	3.2	23.0	2.6	10.1		2.5	8.7	2.9	20.9	2.5	9.7
6		2.7	14.1	3.2	34.6	2.7	15.7		2.2	11.5	2.5	27.0	2.2	12.8
8		2.6	18.1	3.3	47.5	2.7	21.0		2.1	14.6	2.2	31.7	2.1	16.3
10		2.6	22.6	3.3	59.4	2.8	27.2		2.0	17.4	2.1	37.8	2.0	19.4
2	1.0	1.6	2.9	2.3	8.3	1.5	2.9	80	3.1	5.4	3.5	12.6	2.8	5.4
4		2.0	7.0	2.4	17.3	1.9	7.4		2.2	7.7	2.6	18.7	2.2	8.5
6		2.1	11.0	2.4	25.9	2.1	12.2		1.9	9.9	2.2	23.8	2.0	11.7
8		2.2	15.3	2.5	36.0	2.3	17.9		1.8	12.5	2.0	28.8	1.9	14.8
10		2.2	19.2	2.5	45.0	2.3	22.3		1.8	15.7	1.9	34.2	1.8	17.5

Influent to the primary settler

Al/p = 27/31 ≑ 0.871   Fe/p = 55.8/31 = 1.8

Table 4.3 Objective of Dissolved Phosphorus Removal vs. Coagulant Dosage and Mole Ratio

Secondary effluent

P-D in influent (mg/ℓ)	Objective P-D in effluent (mg/ℓ)	$Al_2(SO_4)_3$		$FeCl_3$		MICS		Objective P-D removal (%)	$Al_2(SO_4)_3$		$FeCl_3$		MICS	
		Mole ratio	Al(mg/ℓ)	Mole ratio	Fe(mg/ℓ)	Mole ratio	Al + Fe (mg/ℓ)		Mole ratio	Al(mg/ℓ)	Mole ratio	Fe(mg/ℓ)	Mole ratio	Al + Fe (mg/ℓ)
1	0.3	2.9	2.5	(3.3)	5.9	(3.3)	(3.2)	90	–		–	–	–	
2		3.0	5.2	–	–	(3.2)	(6.2)		(3.5)	6.1	–	–	–	
3		3.0	7.8	–	–	(3.1)	(9.0)		3.0	7.8	–	–	(3.1)	(9.0)
1	0.5	1.9	1.7	2.1	3.8	2.0	1.9	85	–		–	–	–	
2		2.2	3.8	2.7	9.7	2.3	4.5		3.0	5.2	–	–	(3.2)	(6.2)
3		2.4	6.3	3.1	16.7	2.4	7.0		2.6	6.8	(3.3)	17.8	2.5	7.3
1	1.0	0	0	0	0	0	0	80	(3.5)	3.0	–	–	–	
2		1.3	2.3	1.5	5.4	1.4	2.7		2.6	4.5	(3.2)	11.5	2.7	5.2
3		1.6	0.2	2.0	10.8	1.8	5.2		2.3	6.0	2.8	15.1	2.2	6.4

Table 4.4 Comparison between Constant Feed and Mole Ratio Control (Influent flow is constant.)

Mole ratio (Al/p)	Dissolved phosphorus				ⓑ/ⓐ	ⓐ-ⓑ
	Items	Influent	Constant feed ⓐ	Mole ratio control ⓑ		
1	Lo*	7.01	4.46	4.07	0.91	0.39
	Co**	3.36	2.14	1.95		0.19
	Re***	–	36	42	–	-6
2	Lo	7.01	2.38	2.06	0.86	0.32
	Co	3.36	1.14	0.99		0.15
	Re	–	66	71	–	-5
3	Lo	7.01	0.89	0.73	0.81	0.16
	Co	3.36	0.43	0.35		0.08
	Re	–	87	90	–	-3

* Lo: load (kg·P/D)

** Co: average concentration (mg·P/ℓ)

*** Re: removal (%)

Table 4.5 Coefficients of Regression Lines of Fig. 4.9

Dissolved phosphorus in influent	Coefficient	Mole ratio (Al/p)		
		1	2	3
3.36 mg·P/ℓ max.	a*	0.574	0.606	0.452
	b*	-1.984	-2.200	-1.640
	r**	0.884	0.873	0.977
3.36 mg·P/ℓ min.	a	0.110	0.189	0.091
	b	-0.504	-0.738	-0.430
	r	0.311	0.754	0.553

* [(P-D in effluent treated by constant feed) - (P-D in effluent treated by mole ratio controle)]
= a · [P-D in influent] + b

** correlation coefficient

Table 4.6 Comparison between Constant Feed and Mole Ratio Control (Influent flow changes.)

Mole ratio (Al/p)	Dissolved phosphorus						Al dosage		
	Items	Influent	Constant feed ⓐ	Mole ratio control	ⓑ/ⓐ	ⓐ-ⓑ	Constant feed ⓒ	Mole ratio control	ⓓ/ⓒ
1	Lo*	7.48	4.84	4.34	0.90	0.50	6.11	6.52	1.067
	Co**	3.59	2.32	2.08		0.24			
	Re***	–	35	42		-7			
2	Lo	7.48	2.65	2.20	0.83	0.45	12.22	13.04	
	Co	3.59	1.27	1.06		0.21			
	Re	–	65	71		-6			
3	Lo	7.48	1.04	0.72	0.69	0.32	18.33	19.55	
	Co	3.59	0.50	0.35		0.15			
	Re	–	86	90		-4			

* Lo: load (kg·P/D)

** Co: average concentration (mg·P/ℓ)

*** Re: removal (%)

Table 4.7 Comparison between Constant Feed and Mole Ratio Control

Mode ratio in cons-tant feed	Correspondence of influent flow and dissolved phosphorus in influent	Mole ratio in mole ratio control	Dissolved phosphorus				ⓑ / ⓐ	ⓐ-ⓑ
			Item	Influent	Constant feed ⓐ	Mole ratio control ⓑ		
1	Direct proportion****	0.83	Lo*	8.46	5.86	5.30	0.90	0.56
			Co**	4.06	2.81	2.54		0.27
			Re***	–	31	37	–	-6
	Actual correspondence****	0.94	Lo	7.48	4.85	4.50	0.93	0.35
			Co	3.59	2.32	2.16		0.16
			Re	–	35	40	–	-5
	Influent flow is constant	1.00	Lo	7.01	4.46	4.07	0.91	0.39
			Co	3.36	2.14	1.95		0.19
			Re	–	36	42	–	-6
	Inverse proportion****	1.17	Lo	6.00	3.54	3.34	0.94	0.20
			Co	2.88	1.70	1.60		0.10
			Re	–	41	44	–	-3
2	Direct proportion	1.66	Lo	8.46	3.26	2.97	0.91	0.29
			Co	4.06	1.56	1.42		0.14
			Re	–	62	65	–	-3
	Actual correspondence	1.87	Lo	7.48	2.65	2.40	0.91	0.25
			Co	3.59	1.27	1.15		0.12
			Re	–	65	68	–	-3
	Influent flow is constant	2.00	Lo	7.01	2.38	2.06	0.86	0.32
			Co	3.36	1.14	0.99		0.15
			Re	–	66	71	–	-5
	Inverse proportion	2.34	Lo	6.00	1.75	1.52	0.87	0.23
			Co	2.88	0.84	0.73		0.11
			Re	–	71	75	–	-4
3	Direct proportion	2.48	Lo	8.46	1.34	1.38	1.03	-0.04
			Co	4.06	0.64	0.66		-0.02
			Re	–	84	84	–	0
	Actual correspondence	2.81	Lo	7.48	1.04	0.92	0.88	0.12
			Co	3.59	0.50	0.44		0.06
			Re	–	86	88	–	-2
	Influent flow is constant	3.00	Lo	7.01	0.89	0.73	0.81	0.16
			Co	3.36	0.43	0.35		0.08
			Re	–	87	90	–	-3
	Inverse proportion	3.50	Lo	6.00	0.58	0.37	0.64	0.21
			Co	2.88	0.28	0.18		0.10
			Re	–	90	94	–	-4

* Lo: load (kg·P/ℓ)

** Co: average concentration (mg·P/ℓ)

*** Re: removal (%)

**** refer to Fig. 6

Table 4.8 Operation Condition of Alum Precipitation and Filtration

Process	Condition
Influent	Secondary effluent
Alum precipitation	Coagulant $Al_2(SO_4)_3$ Dosage 3.9 mg·Al/ℓ Influent flow 216 m^3/D Overflow rate of alum sedimentation tank 50 $m^3/m^2 \cdot D$
Filtration	Gravity flow, constant head, dual media Initial flow rate 218 m/D Total head loss at filtration end

Table 4.9 Results of Alum Precipitation and Filtration at Yokosuka Pilot Plant

		Influent (secondary effluent)	Effluent from alum sedimentation tank	Effluent from filter
Temp.	(°C)	11.7		
pH		7.22	6.96	7.12
Turbidity	mg/ℓ	27.1	18.2	2.3
	%*		32.7	91.4
SS	mg/ℓ	10.5	6.9	1.3
	%		34.2	87.7
P	mg·P/ℓ	1.782	0.691	0.173
	%		61.2	90.3
P. ortho	mg·P/ℓ	1.309	0.472	0.109
	%		63.9	91.7
$\mathrm{COD}_{\mathrm{cr.}}$	mg/ℓ	45.8	30.6	23.8
	%		33.1	48.1
$\mathrm{COD}_{\mathrm{Mn.}}$	mg/ℓ	14.3	10.4	7.1
	%		27.3	50.6
BOD	mg/ℓ	14.6	6.5	4.5
	%		55.7	69.0
ABS	mg/ℓ	0.192	0.180	0.127
	%		6.3	33.9
Common bact.	N/mℓ	31,000		19,000
	%			38.7
Colo-form bact.	N/mℓ	3,600		340
	%			90.6

* Removal to influent

Table 4.10 Results of Alum Sedimentation Tank Operation

Alum Dosage: 3.9 mg. Aℓ/ℓ
Overflow Rate of Alum Sedimentation Tank: 50 $m^3/m^2 \cdot D$

Item \ Condition		Fe^{+++} addition (mg/ℓ)				Return sludge (% to influent flow)				Polymer addition (mg/ℓ)		
		0	0.5	2.0	5.0	0	10	20	30	0	0.1	0.5
Temp. of influent (°C)		18.7	19.5	20.5	19.9	22.0	21.3	21.0	22.2	24.5	24.6	24.8
pH	① *	7.20	7.34	7.24	7.28	7.24	7.29	7.22	7.42	7.46	7.26	7.36
	② **	6.92	6.98	6.93	6.89	7.03	6.96	6.92	7.09	7.11	7.12	7.08
SS (mg/ℓ)	①	22.8	24.0	31.0	27.0	9.4	11.6	16.8	37.0	4.3	11.7	25.0
	②	13.0	14.5	15.4	13.8	12.7	8.1	13.7	15.1	5.7	12.6	14.4
P (mg/ℓ)	①	1.487	1.906	3.320	2.054	2.538	1.709	2.170	2.020	1.895	1.864	1.156
	②	0.348	0.461	0.663	0.689	0.783	0.695	0.779	0.590	0.118	0.194	0.347
P-D (mg/ℓ)	①	0.836	1.331	2.265	1.666	1.882	1.226	1.569	1.297	1.155	1.647	0.649
	②	0.061	0.105	0.142	0.100	0.270	0.044	0.233	0.076	0.127	0.212	0.065

* ① Influent (secondary effluent) ** ② Effluent from alum sedimentation tank

Table 4.11 Constitution of Filtration Media

		No. 1 filter	No. 2 filter
Anthercite coal	Depth (cm)	30	30
	Effective size (mm)	0.8	1.2
	Uniformity coefficient	1.4	1.4
Silica sand	Depth (cm)	30	30
	Effective size (mm)	0.45	0.55
	Uniformity coefficient	1.2	1.2

Table 4.12 Results of Alum Sludge Dewatering by Centrifuge

Character of Feed Sludge (Thickend Sluge)

Water Content = 98.6%, Aℓ = 131 mg/g·dry sludge
P = 36.8 mg/g·dry sludge, VSS = 463 mg/g·dry sludge

Expe. No.	Experiment condition*			Dewatered sludge		Suspended solids in centrate (g/ℓ)
	Centirfugal force (G)	Sludge feed rate (m^3/h)	Conveyer revolution (rpm)	Rate of total solid captured (%)	Water content (%)	
1	1,000	1.26	6	28.7	86.9	10.5
2	2,000	1.38	6.5	26.2	86.7	9.7
3	3,000	1.40	8	33.8	86.0	9.0
4	2,000	0.46	6.5	48.4	88.2	7.0
5	2,000	0.81	6.5	35.1	86.6	8.7
6	2,000	1.79	6.5	22.5	85.4	10.3

* Pool depth is 1.5~2.4 mm.

Table 4.13 Influent and Effluent Quality of the Highrate Chemical Coagulation Clarification System

Item		Influent		Effluent range
		Range	Mean	
Suspended solid	(mg/ℓ)	12 ~ 32	18	1~10 (2~3 almost)
pH		6.95~ 7.65	7.2	–
M-Alkalinity	(mg/ℓ)	102 ~140	–	–
Total-COD_{Mn}	(mg/ℓ)	7.4 ~ 11.0	9.0	4.8~7.7
Soluble-COD_{Mn}	(mg/ℓ)	5.8 ~ 8.1	6.5	4.3~6.8
Suspended-COD_{Mn}	(mg/ℓ)	1.3 ~ 7.0	2.2	–
Total-BOD_5	(mg/ℓ)	7.6 ~ 14.0	11.2	trace ~3.0
Soluble BOD_5	(mg/ℓ)	2.8 ~ 3.3	3.0	trace ~1.0
PO_4-	(mg/ℓ)	0.8 ~ 1.4	0.9	trace

Table 4.14 Water Quality of Influent and Effluent, and Chemical Dosing Rate

Items	Unit	Type A		Type B		Type C	
		Influent	Effluent	Influent	Effluent	Influent	Effluent
Suspended solid	mg/ℓ	18 ~ 47	2 ~14	13 ~145	4 ~ 25	19 ~248	~15
pH		7.15~ 7.6	6.4~ 6.8	7.15~ 7.58	6.3~ 6.7	7.25~ 7.5	6.3~ 6.8
M-Alkalinity	mg/ℓ	140 ~270	–	117 ~310	–	143 ~270	–
Total COD_{Mn}	mg/ℓ	20 ~ 40	9.5~22.5	21 ~106	10 ~ 36	19 ~ 74	8 ~30
Soluble COD_{Mn}	mg/ℓ	10 ~ 27	7.6~18.4	15 ~ 40	10 ~ 25	13 ~ 39	11.1~15.5
Total COD_{cr}	mg/ℓ	80 ~166	67 ~96	144 ~197	96 ~108	–	–
PO_4	mg/ℓ	3.3 ~ 11	0.5~ 1.6	1.8 ~ 14	0.9~ 1.4	–	–
BOD_5	mg/ℓ	18 ~ 30	3 ~ 6	20 ~ 30	4 ~ 22	106	30.5
Alum dosing rate	mg/ℓ	60~80		90~130		80~140	
Polymer dosing rate	mg/ℓ	1~1.2		1~1.2		1~1.2	

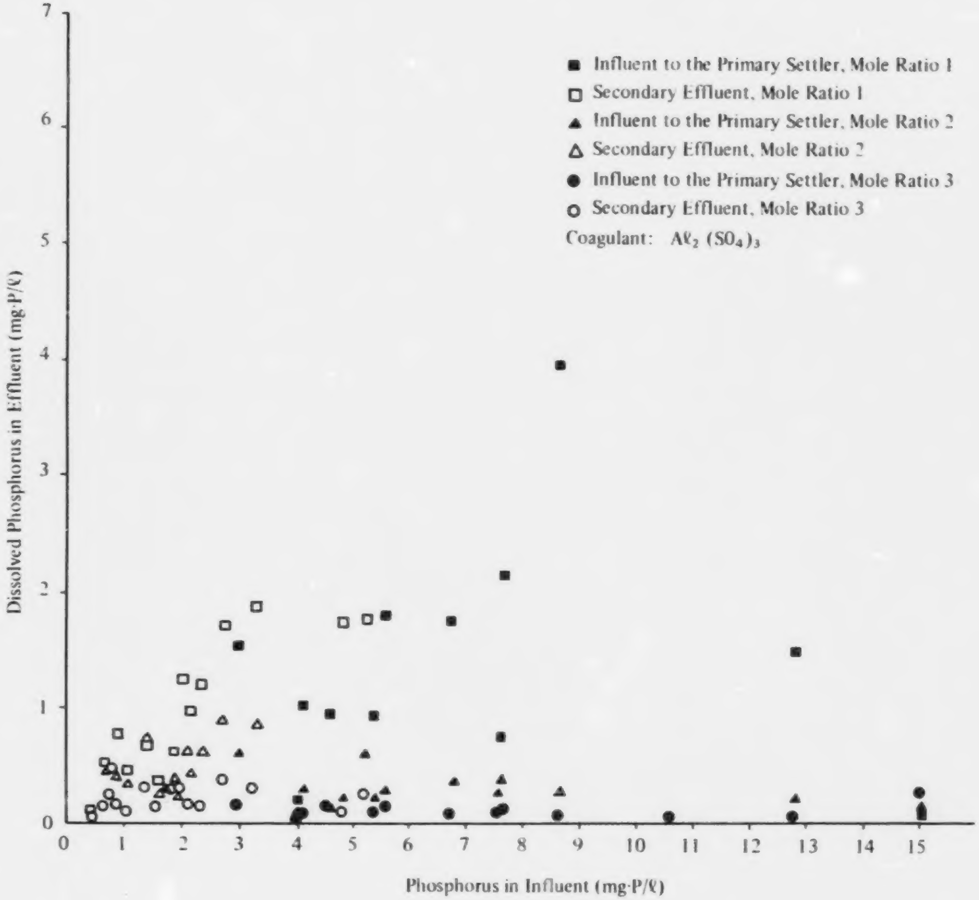


Fig. 4.1 **Phosphorus in Influent vs. Dissolved Phosphorus in Effluent (Mole ratios are based on phosphorus in influent.)**

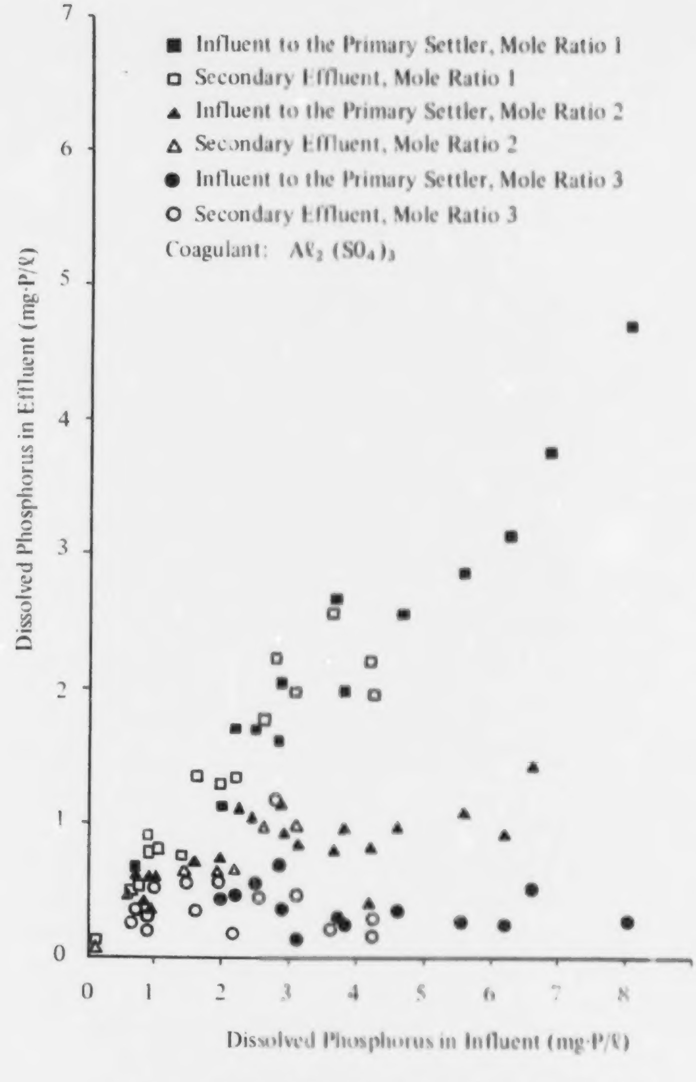


Fig. 4.2 Dissolved Phosphorus in Influent vs. Dissolved Phosphorus in Effluent (Mole ratios are based on dissolved phosphorus in influent.)

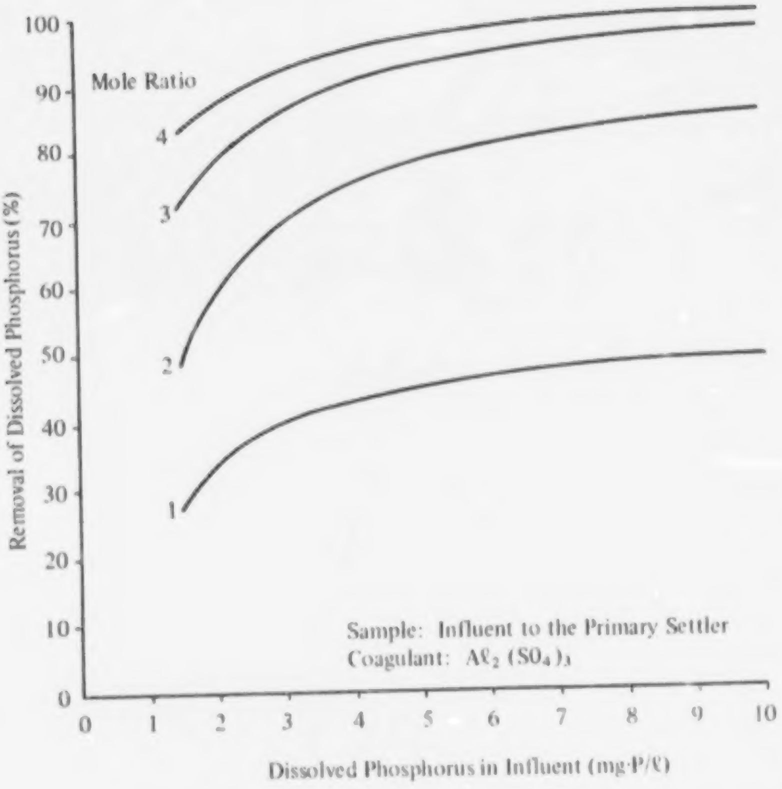


Fig. 4.3 Dissolved Phosphorus in Influent vs. Removal of Dissolved Phosphorus

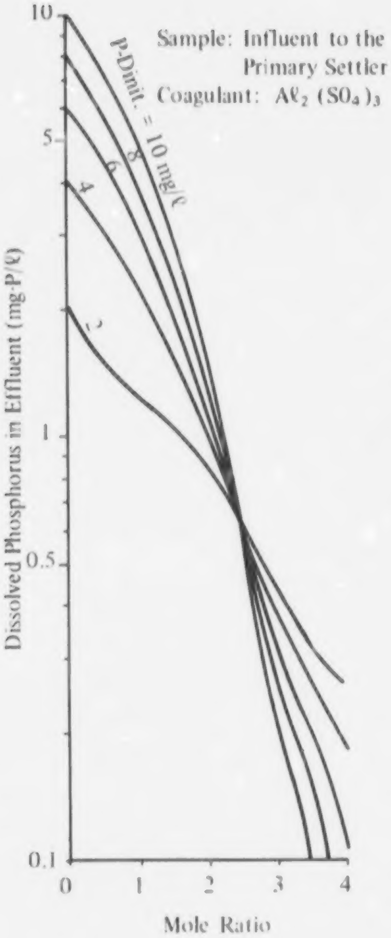


Fig. 4.4 Mole Ratio vs. Dissolved Phosphorus in Effluent

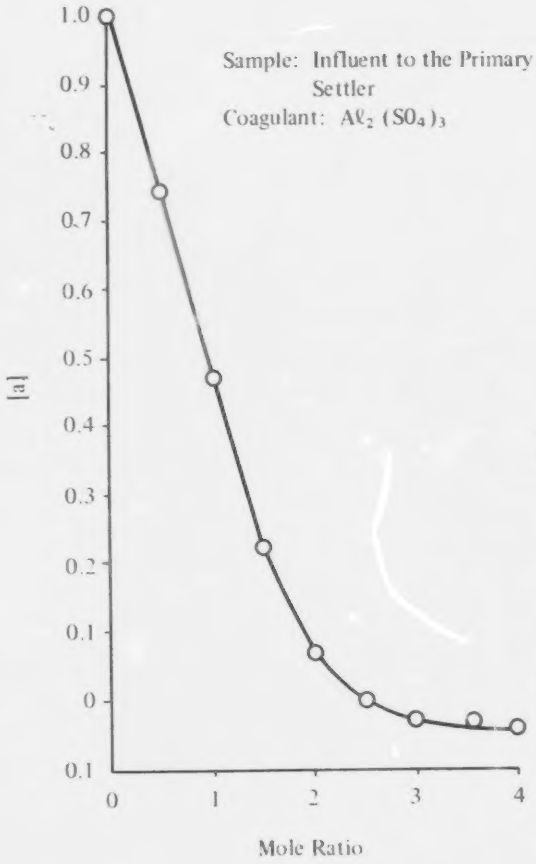


Fig. 4.5 **Mole Ratio vs. a**

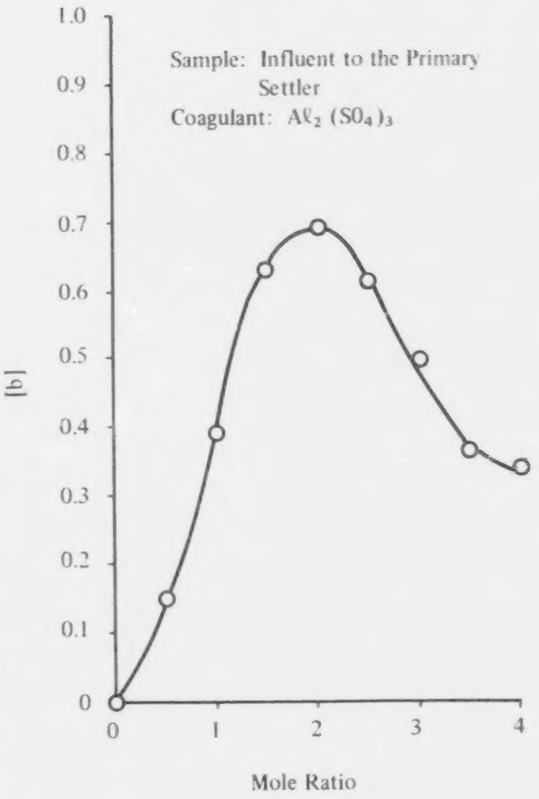


Fig. 4.6 **Mole Ratio vs. b**

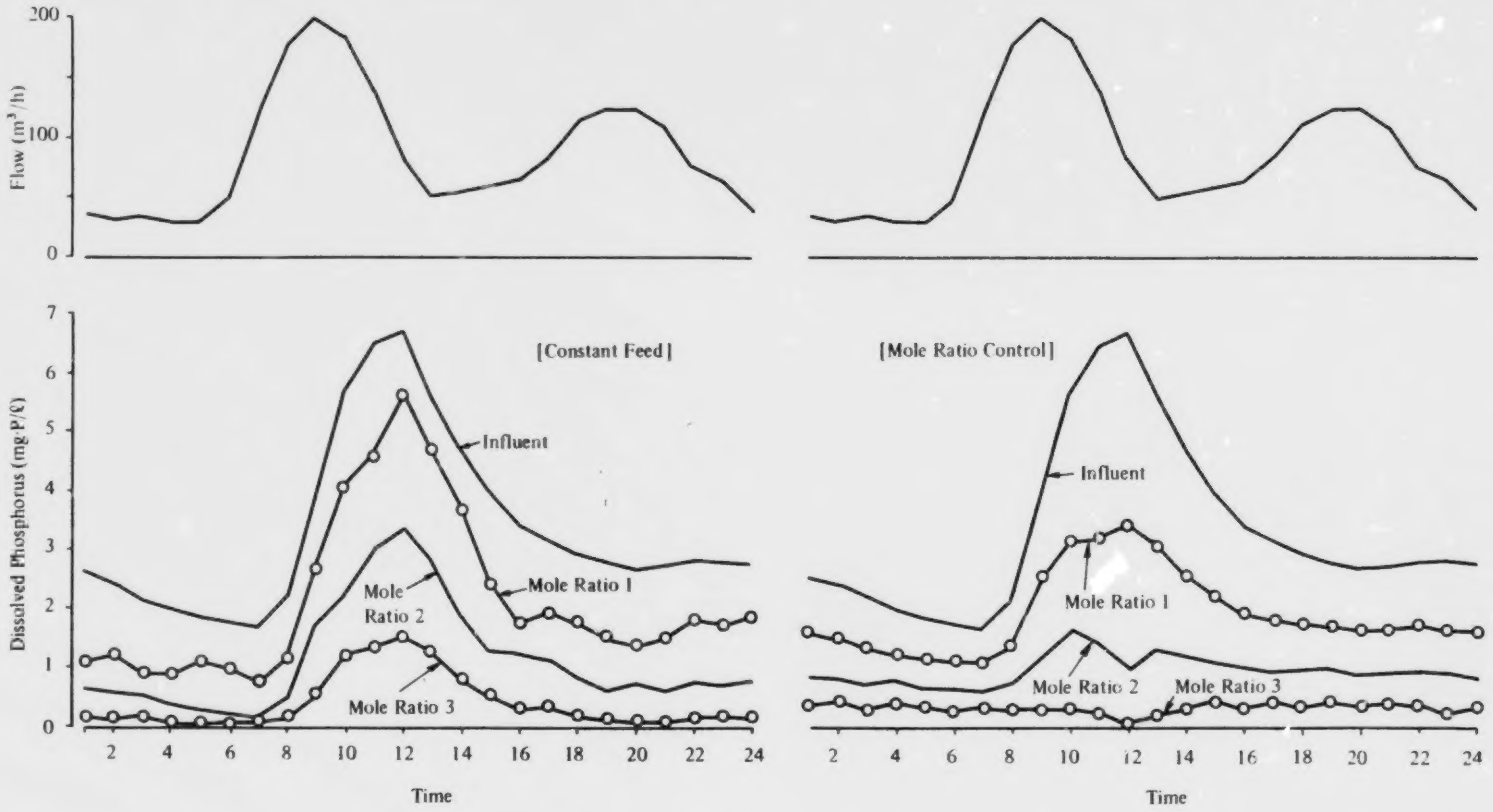


Fig. 4.7 Relationship between Influent Flow, Dissolved Phosphorus Concentration, and Time

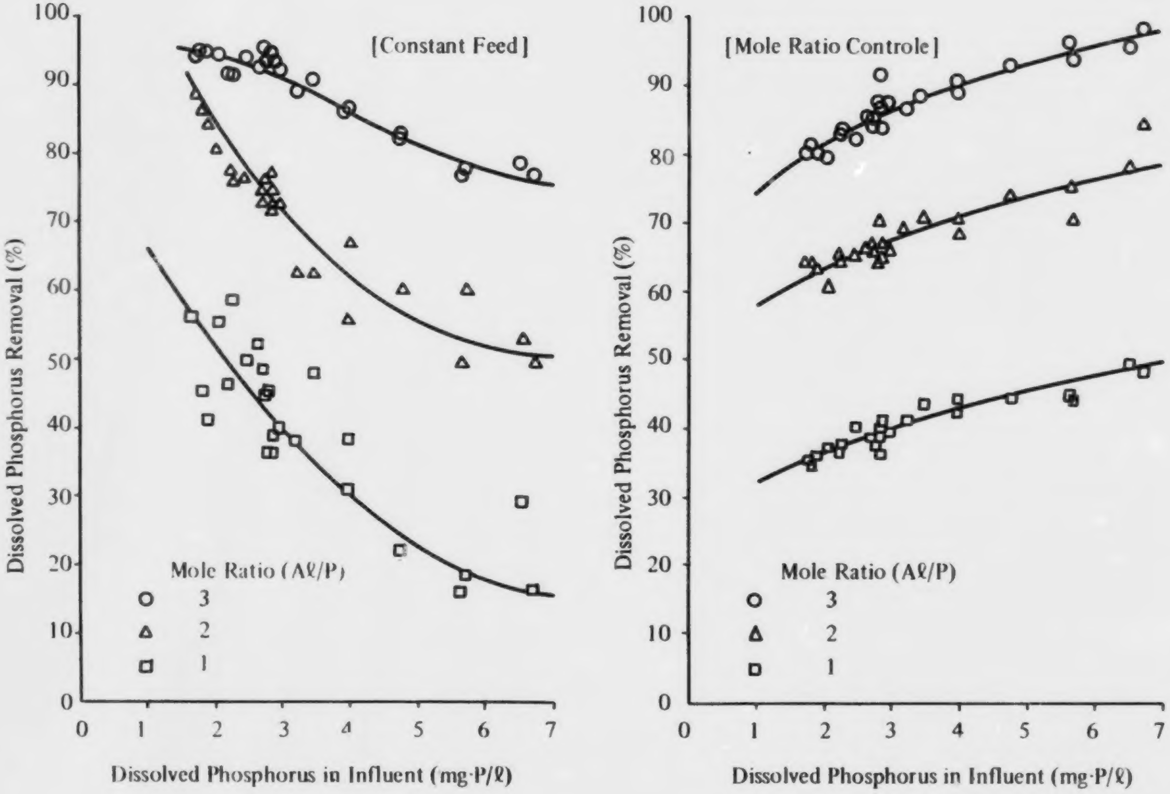


Fig. 4.8 Dissolved Phosphorus in Influent vs. Dissolved Phosphorus Removal

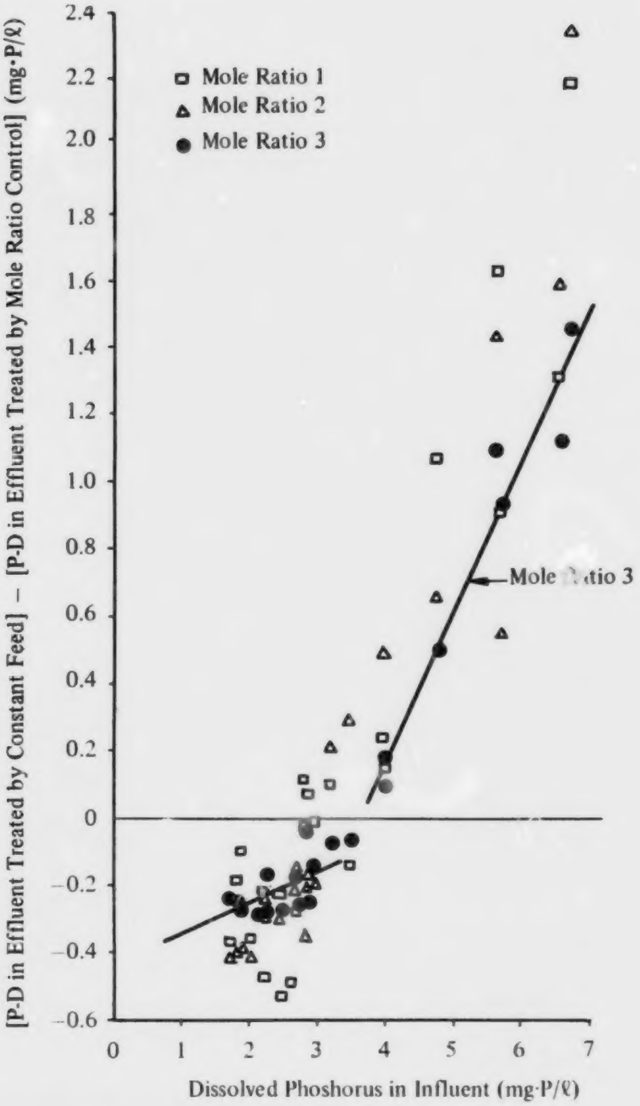


Fig. 4.9 Dissolved Phosphorus in Influent vs. Difference of Dissolved Phosphorus in Effluent between two Coagulant Feed Methods

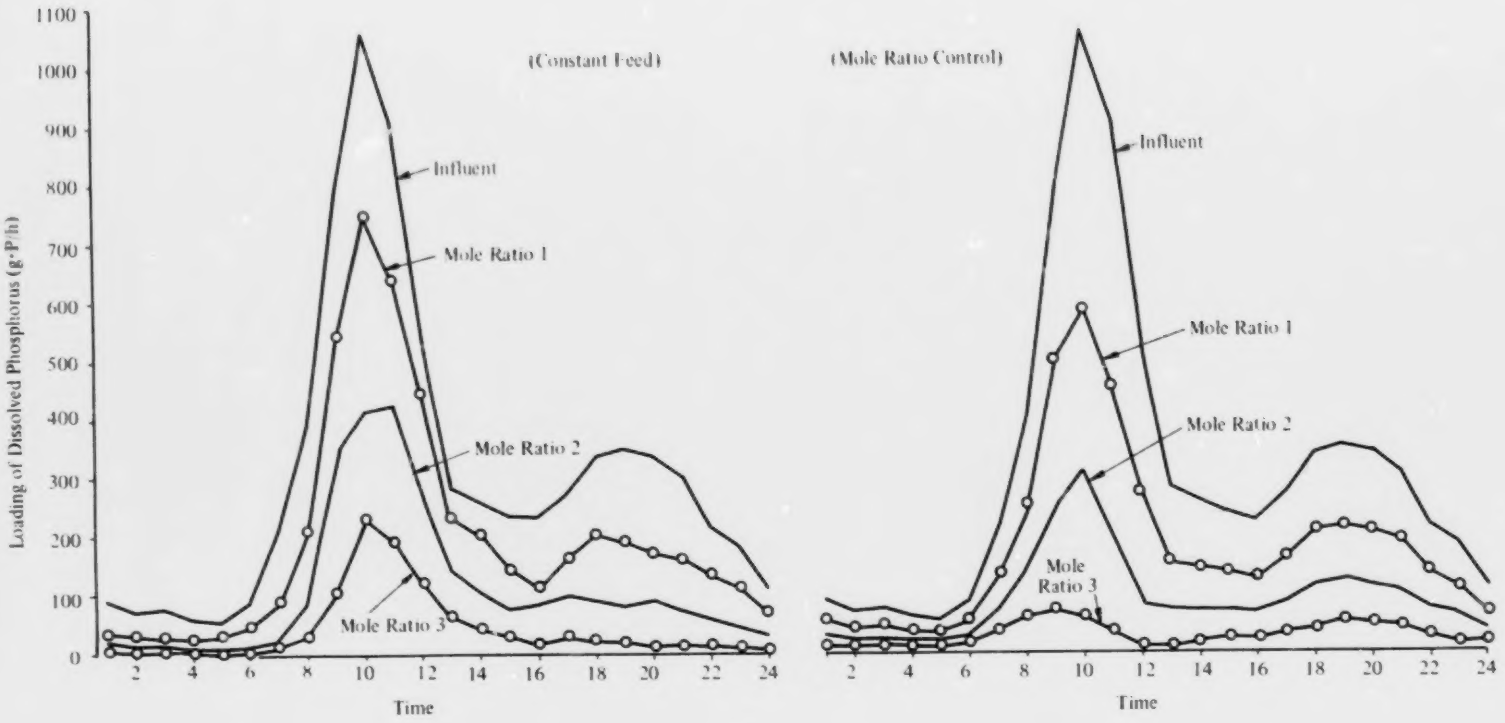


Fig. 4.10 Relationship between Loading of Dissolved Phosphorus and Time

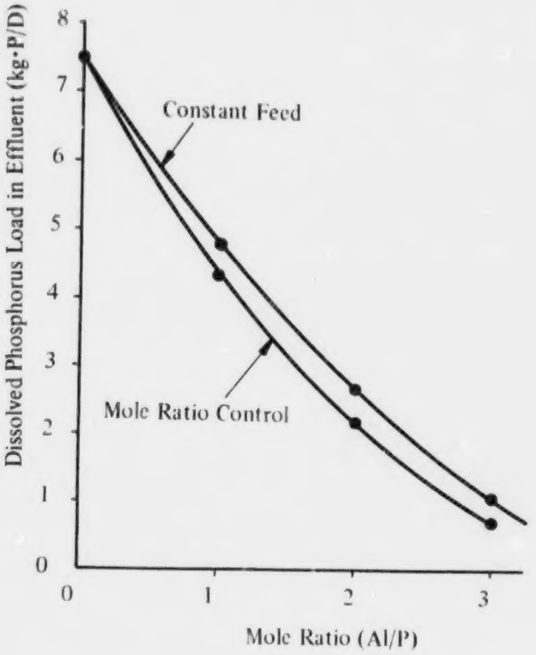
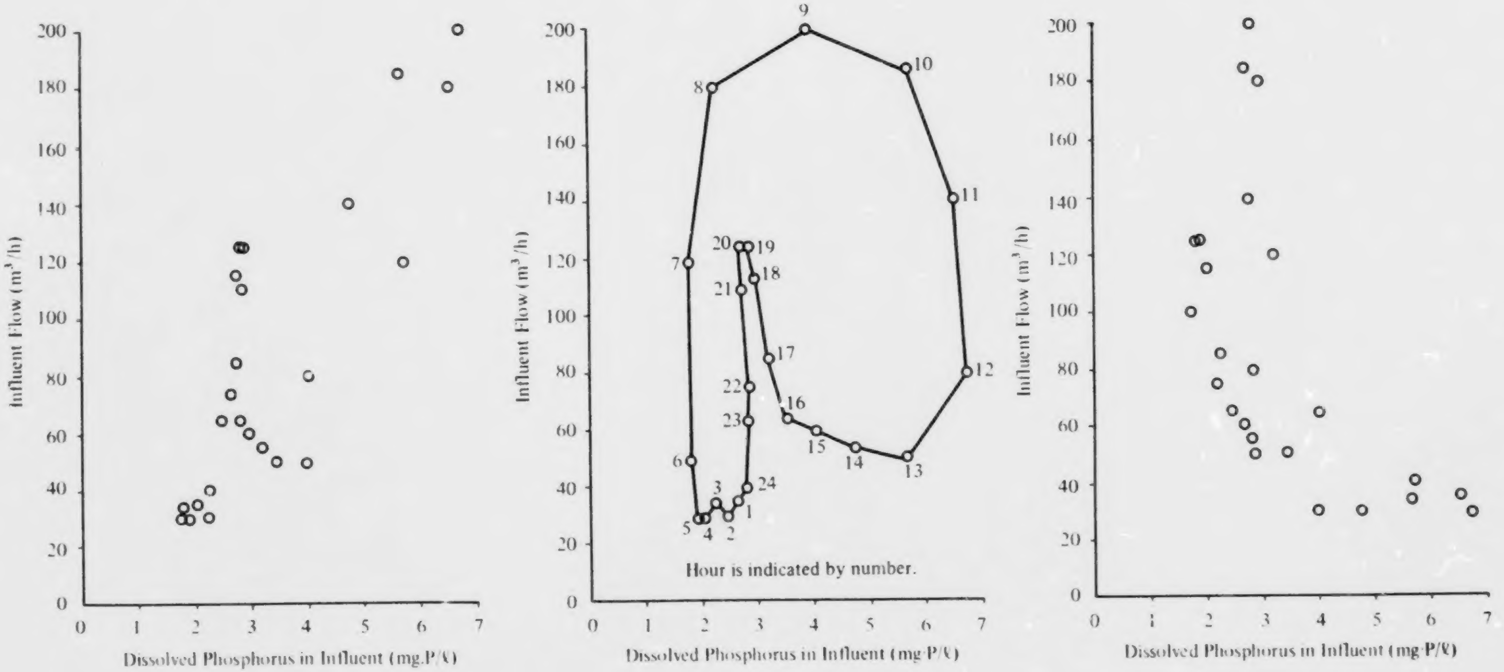


Fig. 4.11 Mole Ratio vs. Dissolved Phosphorus Load in Effluent



(Influent Flow is in Direct Proportion to Phosphorus)

(Correspondence of Influent Flow and Phosphorus is Actual)

(Influent Flow is in Inverse Proportion to Phosphorus)

Fig. 4.12 Relationship between Dissolved Phosphorus and Influent Flow Used for Caluculation

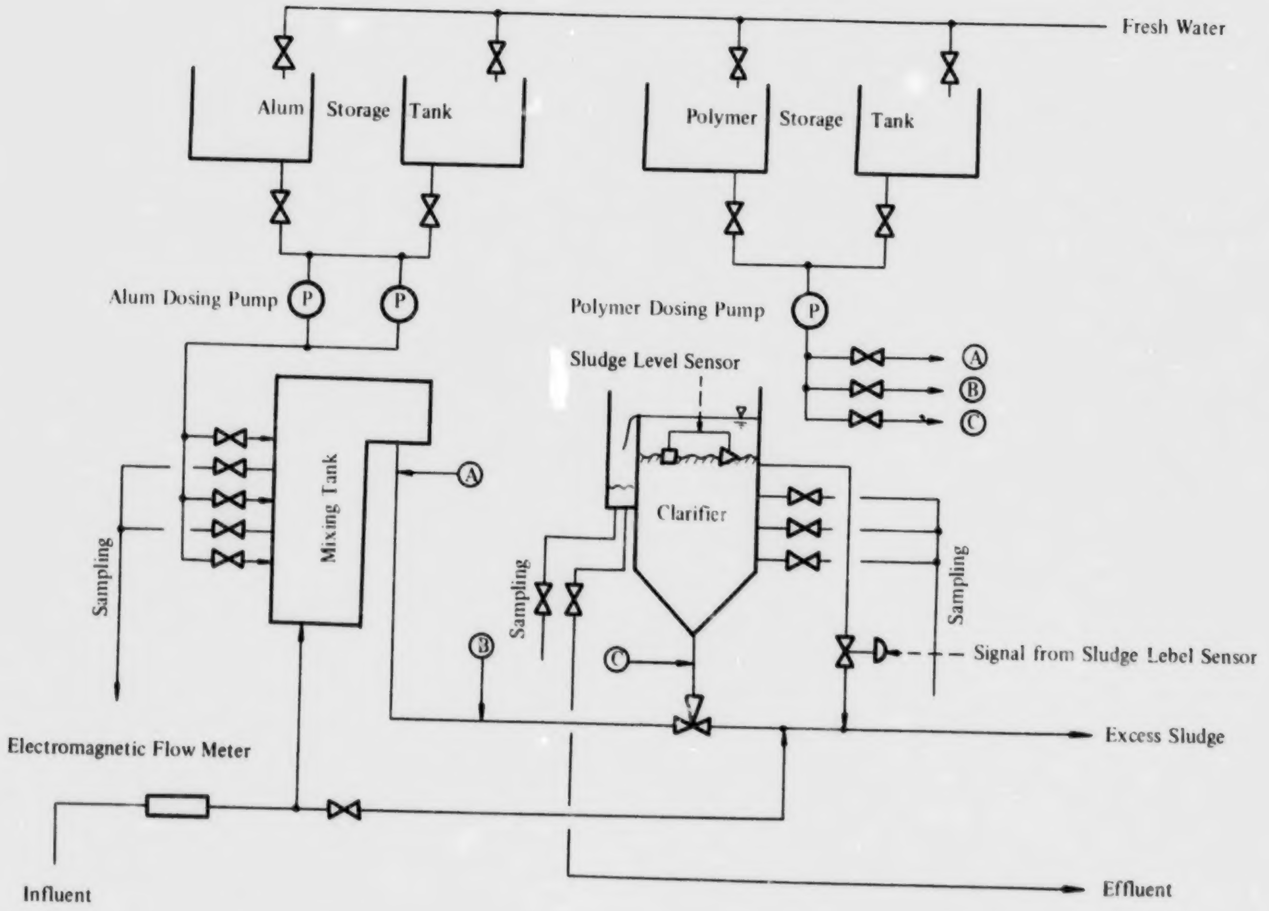

Fresh Water
Alum Storage Tank
Polymer Storage Tank
Alum Dosing Pump
P
P
Polymer Dosing Pump
P
A
B
C
Sludge Level Sensor
Mixing Tank
A
Clarifier
Sampling
Sampling
Sampling
B
C
Signal from Sludge Lebel Sensor
Excess Sludge
Electromagnetic Flow Meter
Influent
Effluent

Fig. 4.13 Flow Diagram of High Rate Chemical Coagulation and Clarification

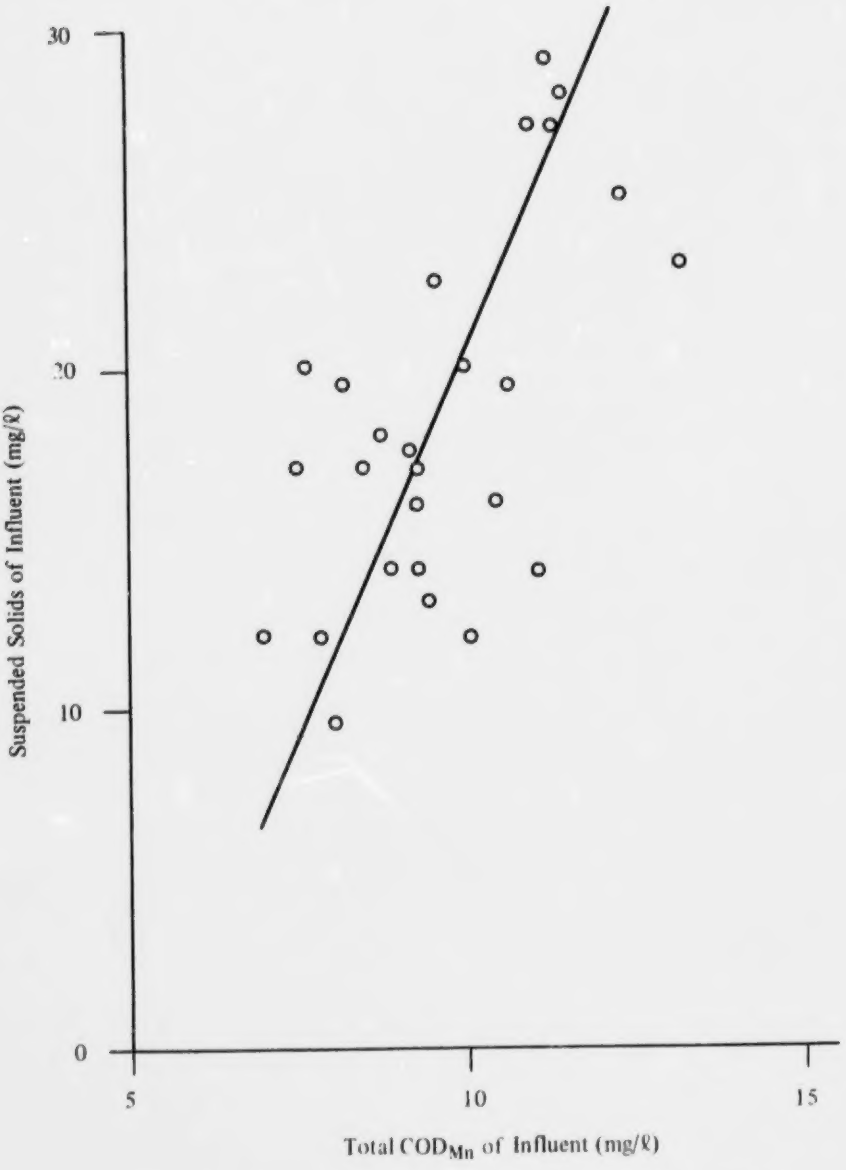


Fig. 4.14 Corelation between Suspended Solids and Total COD_{Mn}

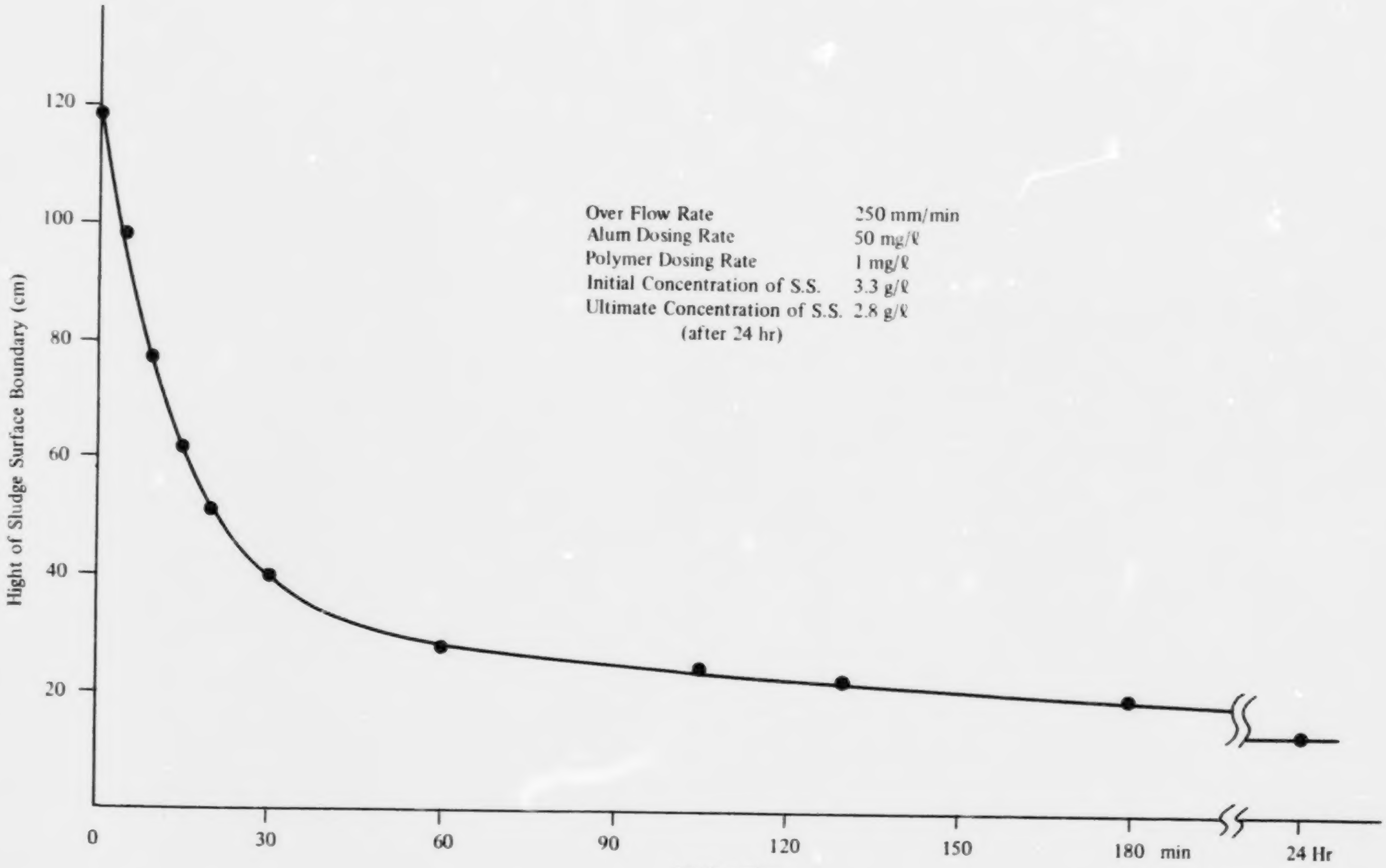


Fig. 4.15 Settability of Excess Sludge

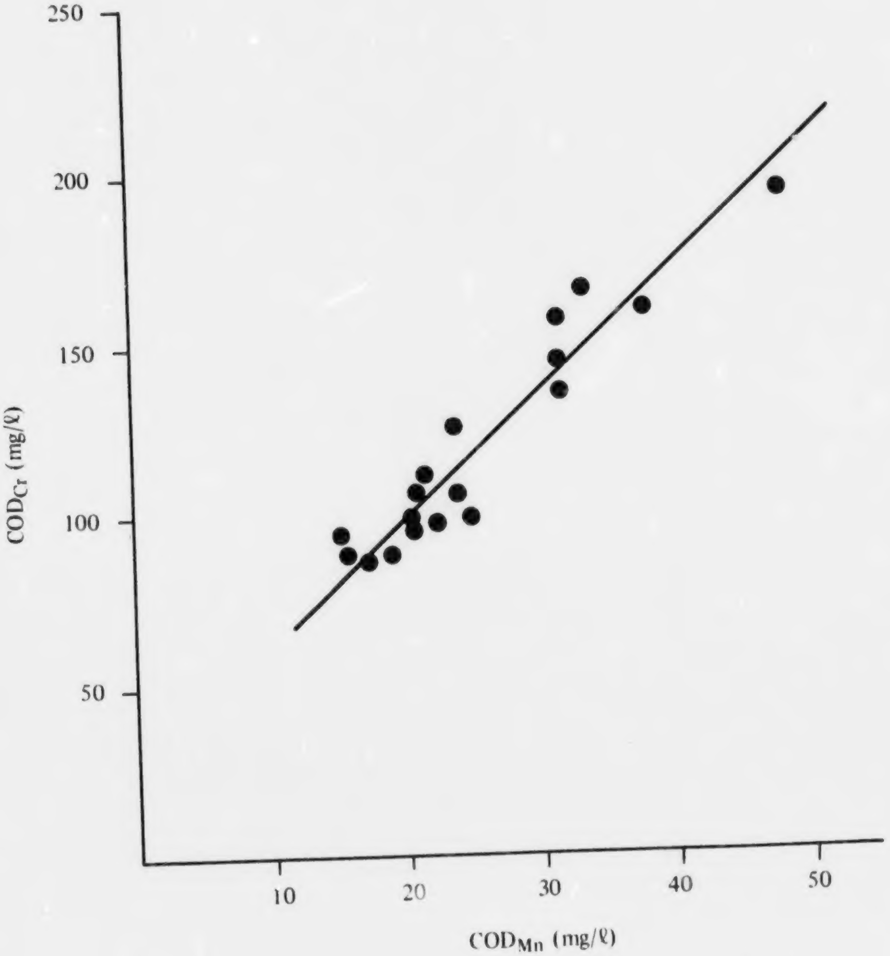


Fig. 4.16 Corelation between COD_{Mn} and COD_{Cr} of Influent

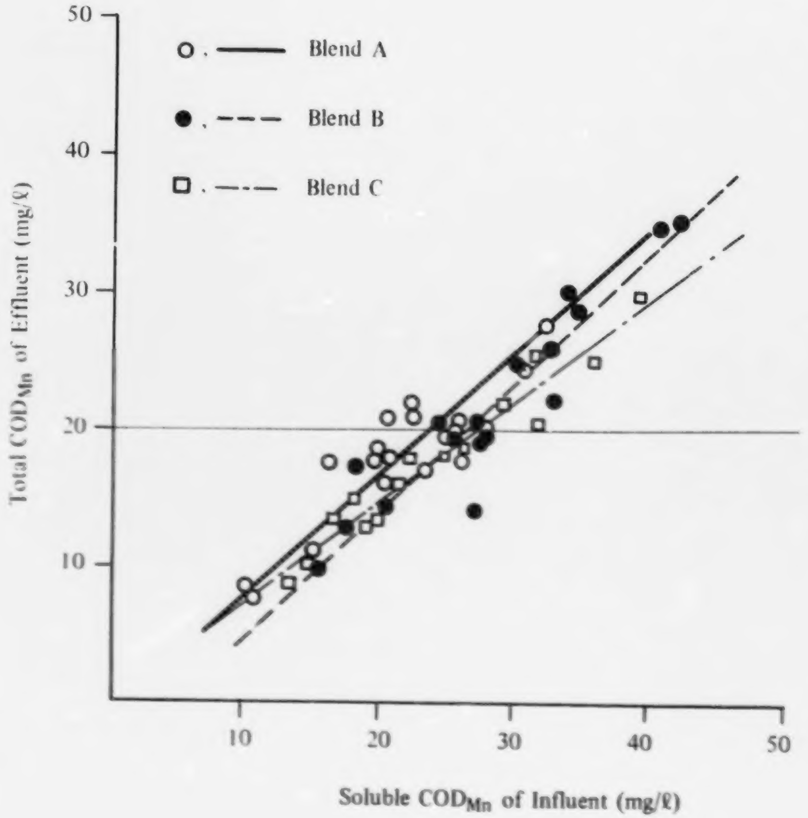


Fig. 4.17 Corelation of Inf. Solube COD_{Mn} and Eff. Total COD

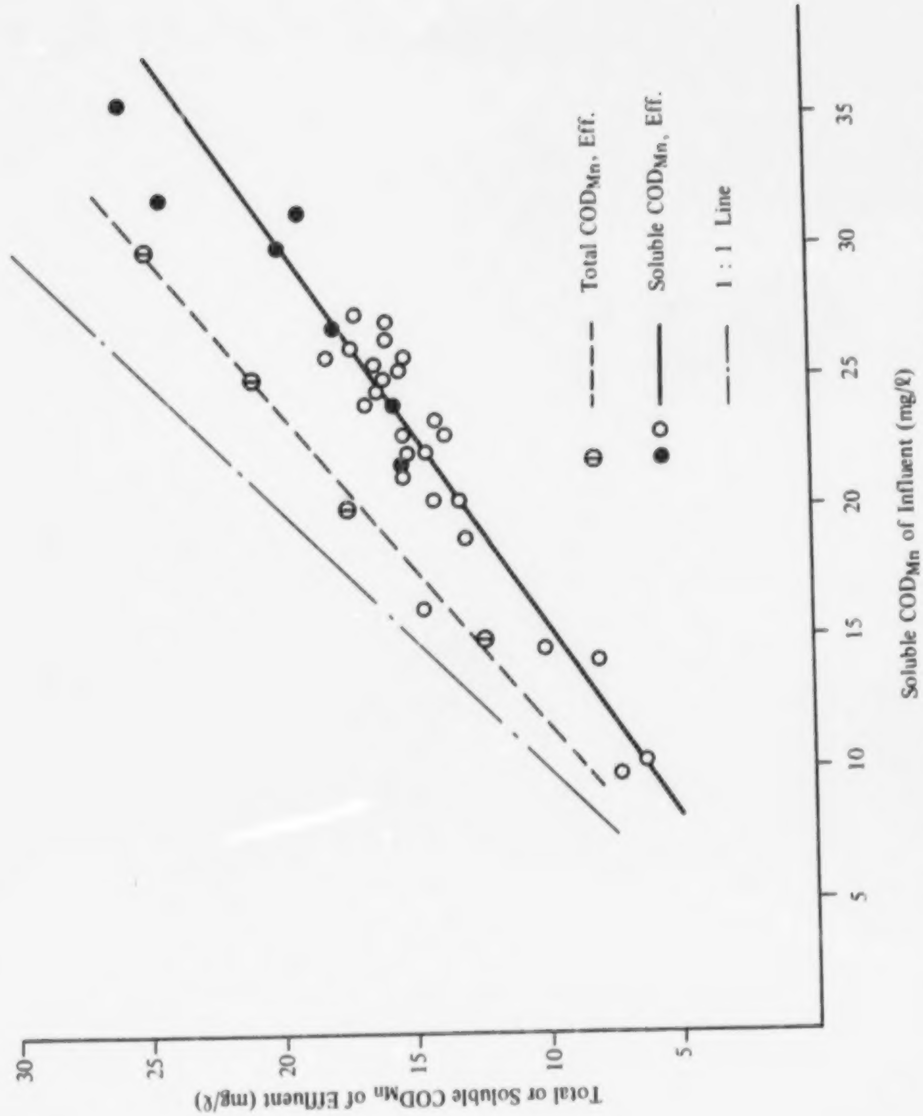


Fig. 4.18 Corelation between Total or Solube COD$_{Mn}$ of Effluent and Solube COD$_{Mn}$ of Influent

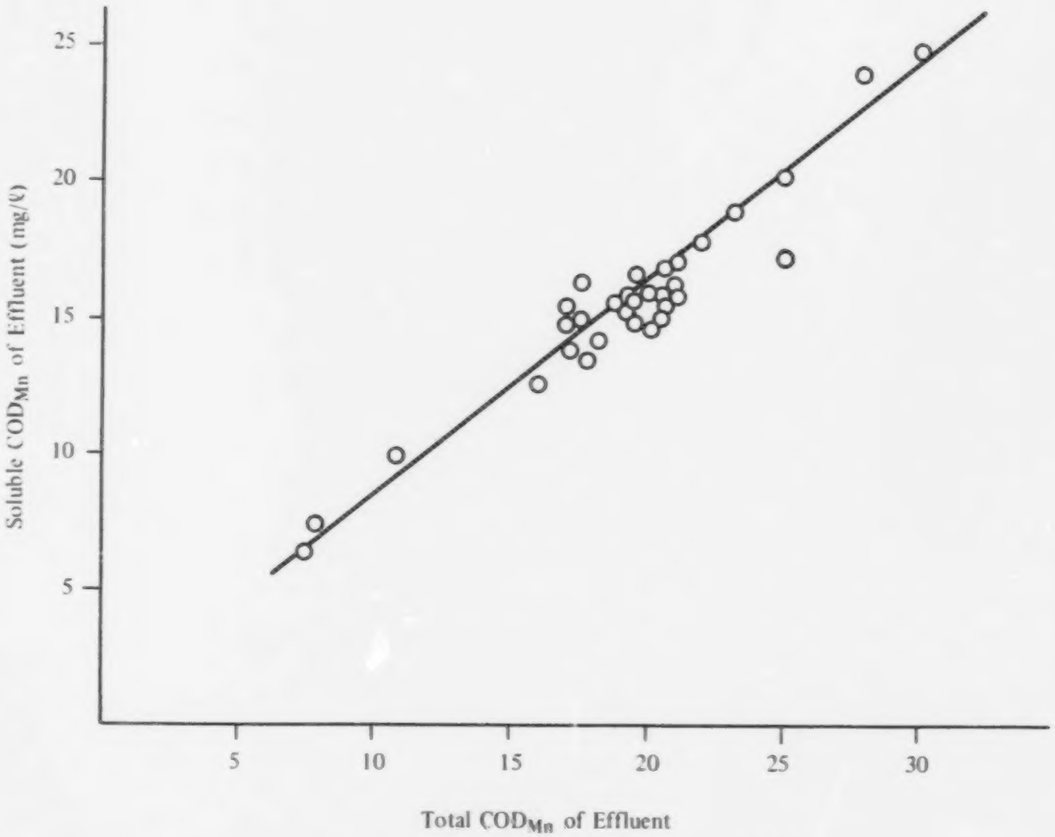


Fig. 4.19 Corelation between Total COD_{Mn} and Soluble COD_{Mn} of Effluent

CHAPTER 5. LIME PRECIPITATION AND RECOVERY OF CALCIUM CARBONATE

5.1 Results of Precipitation 206
  5.1.1 Lime Slaking 206
  5.1.2 Results of Lime Sedimentation Tank Operation 207
  5.1.3 Phosphorus Resolubilization from Floc Containing Phosphorus 207
5.2 Experiment on Scale Formation of Calcium Carbonate 209
5.3 Recovery of Calcium Carbonate from Lime Sludge 210
  5.3.1 Results by Centrifuge 210
  5.3.2 Results of Recarbonation of Lime Sludge 212
  5.3.3 Summary 213

5. LIME PRECIPITATION AND RECOVERY OF CALCIUM CARBONATE

The results of lime precipitation at the Yokosuka Pilot Plant were reported at the Third U.S.-Japan Conference on Sewage Treatment Technology. This report is a summary of experimental results after the Conference. These experiments have the following three points which are different from former ones:

a. Quick Lime instead of slaked lime is used for making line slurry.

b. Rapid mixing tank, flocculation tank and lime sedimentation tank are connected by open channels instead of piping. Before the modification, due to scaling of calcium carbonate in piping, roughness increased and cross sectional area decreased, and thus, water levels in tanks became high, and sometines causing overflow of water from tanks.

c. Sludge thickener and centrifuge are added.

5.1 RESULTS OF PRECIPITATION

5.1.1 LIME SLAKING

Lime slaking is performed as follows: powder quick lime (residue on a 88-micron mesh sieve is less than 5%) is stocked in the hopper; a fixed quantity of quick lime is continuously raked out with a volumetric feeder mounted at the bottom of the hopper; the quick lime is sent to the slaking tank by a screw conveyer; the water level in the slaking tank is kept constant by a hold-up valve. Under the conditions of this experiment, i.e., $9m_3/h$ of influent flow, 300mg; $Ca(OH)_2/l$ of lime dosage and 10% of the concentration of lime slurry, the retention time in the slaking tank is 60 minutes. Problems in slaking operations so far are summed up as follows:

a. Average pH value after lime is added to sewage was 10.3 during the period that slaked lime was used and 10.5 during the period slaked quick lime was used ($Ca(OH)_2$ dosage, 300mg/l). From this, it is considered that use of slaked quick lime creates no problem in terms of pH values.

b. Impurities contained in quick lime (almost all of them is sand) accumulate in the slaking tank. Purity of quick lime is about 95%. Because of this, clogging of the lime slurry feed pump occurred. Therefore, the slaking tank is drained once a day. Drained slurry is returned to the rapid mixing tank.

c. The bottom of the quick lime hopper slants 45 degrees and is connected to the volumetric feeder. This slanting and a vibrator were provided in an attempt to prevent bridge of quick lime in the hopper but they were not sufficient. The vibrator caused quick lime to drop in bulk at a time, thereby changing the apparent specific gravity of quick lime being supplied to the volumetric feeder. To avoid this, once every two hours, a stick is used to poke the hopper from its upper portion to prevent bridge.

d. Calorific value when quick lime is slaked is 15.5kcal/g.mol. Under the above-mentioned conditions for slaking, temperature rise of 21°C is expected in the slaking tank. Thus, if water supply to the slaking tank is suspended due to some reasom or other, only quick lime accumulates in the tank and temperature in the tank goes up. To prevent this a teperature warning device is provided in the slaking tank, which acts to suspend supply of quick lime when the temperature reaches 65°C. But thus far this device has not given a warning: temperature increase of lime slurry flowing out of the slaking tank has been less than 10°C.

5.1.2 RESULTS OF LIME SEDIMENTATION TANK OPERATION

Table 5.1 shows average values of the results of the lime sedimentation tank operation during the period between November, 1973 and March, 1975. Under the operational conditions which are also indicated in the Table, the settling efficiency of the sedimentation tank did not show significant changes. Therefore, from April, 1975, the conditions have been changed as follows: $FeCl_3$ = 0 to 5.0mg/l in terms of Fe; return sludge ratio = 0 to 30%; anionic polymer dosage = 0 to 1.0mg/l; and overflow rate = 10 to $100m^3/m^2$.d. Also, a shallow setting device will be instaled. Effects of these changes will be examined. Table 5.2 shows data obtaned up to now. From these data, it seems that additions of Fe, and polymer and return of sludge do not contribute to improvement in the removal of suspended solids. But it is observed that through return of sludge the concentration of dissolved phosphorus in overflow from the lime sedimentation tank decreases, thereby reducing the phosphorus concentration.

5.1.3 PHOSPHORUS RESOLUBILIZATION FROM FLOC CONTAINING PHOSPHORUS

From 9 a.m., August 20 to 9 a.m., August, 21, 1974, a 24-hour survey was conducted at the Yokosuka Pilot Plant, at the lime dosage of 300mg $Ca(OH)_2$/l. As a result, the phenomenon of phosphorus resolubilization from phosphorus-containing floc that is carried over from the sedimendation thank was confirmed.

Table 5.3 shows average values of pH of effluent from each unit process, phosphorus concentration and dissolved phosphorus concentration that were obtained in this survey. As seen in the Table, while the concentration of phosphorus in the effluent from the lime sedimentation tank is 0.514mg-p/l, dissolved phosphorus concentration is 0.044mg/l, only about 8% of total phosphorus. But as sewage flows down through ammonia stripping, recarbonation and filtration, the percentage of dissolved phosphorus in phosphorus increases sharply, 44%, 90% and 99%, respectively and dissolved phosphorus, which is 0.044mg-p/l in overflow from the lime sedimentation tank, becomes 0.350mg-p/l in effluent from filter, about 8 times of the former. It is considered that this is because decrease in pH values in the processes after the sedimentation tank causes resolubilization of phosphorus from floc containing phosphorus that is carried over from the sedimentation tank.

To confirm this phenomenon, the following experiment was conducted. About 5mg/l of KH_2PO_4 as P was added to distilled water to make acid solution of orthophosphate; 200mg.$Ca(OH)_2$/l of lime was added to this orthophosphate solution and the mixture was mixed for 10 minutes. After mixing, pH was lowered using hypochloric acid. Imediately after pH ajustment, the solution was again mixed and samples were taken. Samples were also taken 90 minutes and 180 minutes after pH adjustment with keeping the sotution mixed. pH and dissolved phosphorus of these samples were analyzed and the results are shown in Fig. 5.1. The Figure indicates that phosphorus is resolved as pH is lowered and that resolubilization occurs in a short period of time.

As stated in the above, in the lime treatment system consisted of lime precipitation – (ammonia stripping) – recarbonation – filtration, phosphorus is resolubilized as pH decreases from floc containing phosphorus that is carried over from the lime sedimentation tank. If two-stage recarbonation is applied, non-dissolved phosphorus may settle more in the calcium carbonate sedimentation tank. However, it is a reasonable assumption that pH of the influent to the calcium carbonate sedimentation tank is 9.5 to 10.0, in which range pH of the effluent from the ammonia stripping tower falls as shown in Table 5.3. The dissolved phosphorus concentration in the ammonia stripping tower effluent was 0.181mg-p/l, which is about four times higher than that in the effluent from the lime sedimentation tank, i.e., 0.044mg-p/l. Thus, not only when single-stage recarbonation is employed but also when two-stage recarbonation is applied, the dissolved phosphorus concentration in the filter efflueut becomes higher than that in the effluent from the lime sedimentation tank, it there exists floc contatining phosphorus carried over from the lime sedimentation tank. Therefore, there arises the possibility that phosphorus removal capacity of lime precipitation is not displayed sufficiently.

To prevent this, two measures are now being examined:

a. Method for improving the removal of suspended solids in the lime sedimentation tank
   See 5.1.2.

b. System in which effluent from the lime sedimentation tank is directly filtered and floc carried over is removed, and then water is introduced to such processes as ammonia stripping and recarbonation.

The latter method might have a problem: calcium carbonate scale is produced in the media, etc. in the filter and interferes operation of the filter. Taking this problem into consideration, and on the basis of observations as to forming of calsium carbonate scale stated in Section 5.2, the following experiments on direct filtration of the effluent from the lime sedimentation tank are schemed.

a Study on receiving methods of influent to the filter to prevent scale forming; contact between influent and air is avoided as much as possible.

b Study on the constitution of filtration media.

c Study on filter washing period, pH of washing water and washing rate.

5.2 EXPERIMENT ON SCALE FORMATION OF CALCIUM CARBONATE

The followings are the summary of observations so far obtained at the Yokosuka Pilot Plant as to scale produced in the lime precipitation process:

a. 90% of the scale is calcium carbonate.

b. Quantity of scale formation in a lower temperature is more than that in a higher temperature.

c. Scale in the earlier stage of formation is relatively soft and can be removed by flashing of water.

d. As a results of experiments with stainless steel, steel, rubber, PVC and wood placed in the rapid mixing tank, the lime sedimentation tank and the lime sedimentation effluent holding tank, difference in materials has little effect on scale formation (Refer to Table 5.4).

e. Quantity of scale formation was greater in the lime sedimentation effluent holding tank in which effluent from the lime sedimentation tank falls down together with air than in the lime sedimentation tank. Quantity of scale formation in the former was 2 to 7 times more than that in the latter (See Tables 5.4 and 5.5).

f. Quantity of scale began to increase sharply after 10 days (See Table 5.5).

5.3 RECOVERY OF CALCIUM CARBONATE FROM LIME SLUDGE

At the Yokasuka Pilot Plant, in order to recover calcium carbonate in lime sludge for reuse of lime, two processes were examined; one was that lime sludge is put into a centrifuge for selective recovering of calcium carbonate contained in lime sludge into dewatered sludge, and the other was that lime sludge is put to recarbonation in order to refine the purity of calcium carbonate by dissolving away impurities.

The raw sewage running into the Shitamachi Sewage Treatment Plant, in Yokosuka in which the pilot plant is constructed contains sea water, and accordingly is rich in magnesium ions, seriously affecting lime treatment.

Magnesium ion is precipitated as magnesium hydroxide, which are highly detrimental to settlability, thickening rate and dewaterability of sludge.

Therefore, of the experiments discussed here, the centrifugal dewatering was an example of low efficiency, and recarbonization of lime sludge an example of so much high efficiency as magnesium hydroxide was abundant.

5.3.1 RESULTS BY CENTRIFUGE

a) Results of laboratory tests

The thickened lime sludge obtained from the Yokosuka Pilot Plant was dewatered in a laboratory centrifuge, and thus the dewatered sludge was separated into three layers as shown in Fig. 5.2.

Layers were taken out one by one, and dried. After a drying, their specific gravity, and grain size distribution were measured: the specific gravity and grain size were recorded largest in the lower layer, followed by medium layer and upper layer in turn. (See Figs. 5.2 and 5.3). The analytical results of composition of each layer are shown in Table 5.6.

According to Table 5.6, about 83% of calcium carbonate in the thickened lime sludge is concentrated in the lower layer, and the ratio of calcium carbonate in the dry sludge is increased up to 75.3% in the lower layer as against 60% in feed sludge. On the other hand, in view of the reuse of lime, the impurities of $Mg(OH)_2$, $Fe(OH)_3$, and $Ca_5(OH)(PO_4)_3$ are increasing more and more, the higher the level becomes; in the lower layer, impurities are about 50% of those in the top or middle layer. Also, the lower the layer, the smaller the water content in the sludge.

Hence, the following conclusions are reached.

1) Lime sludge dewatered by the laboratory centrifuge is separated into three layers. This is because the specific gravity and grain size in the bottom layer are larger than those in the top and middle layers.

2) Taking the lime sludge as being composed of calcium carbonate and impurities, there is tendancy that calcium carbonate concentrates in the bottom layer, while impurities are concentrated in the top and middle layers.

3) The water content in the bottom layer is smaller as compared with that in the top and middle layers.

In short, calcium carbonate in the lime sludge is inferred to have larger grain size and larger specific gravity than impurities and to be excellent in dewaterability.

The selective recovery of calcium carbonate by means of centrifuge is based on this principle: the bottom layer shown in Fig. 5.2 is taken out as dewatered sludge, and impurities are conveyed into supernatant. In this way, relative pure calcium carbonate is concentrated as much as possible into dewatered sludge and at the same time the sludge is dewatered.

b) Results of tests using the centrifuge for the pilot plant

The centrifuge used for the tests is the modified one for the tests which is the smallest, available in the market primarily for sewage sludge dewatering purposes. (standard sludge feed rate: 1 m^3/hr)

Its specifications are as follows.

Bowl diameter:	200ϕ (mm)
Bowl speed:	0 ~ 6,000 rpm
Centrifugal force:	0 ~ 4,000 G
Sludge feed rate:	0 ~ 1.85 m^3/hr
Converyor revolution:	Variable (by stepless speed changer)
Pool depth:	No. 1 (0.5 ~ 0.8 mm), No. 2 (1 ~ 1.6 mm), No. 3 (1.5 ~ 2.4 mm)

The lime sludge used for the tests was the thickened sludge taken from the thickening tank in the pilot plant. Its characteristics, are shown in Fig. 5.4.

For the tests, one out of the four factors–sludge feed rate, centrifugal force, pool depth and conveyor revolution–was taken as a variable while all the others were fixed constant. In this way, the effects of each factor upon the ratio of $CaCO_3$ captured as dewatered sludge, ratio of $CaCO_3$ in dried sludge, and water content of dewatered sludge were investigated. The test results are shown in Fig. 5.4.

The following are found from Fig. 5.4.

1) By increasing the sludge feed rate, the absolute volume of $CaCO_3$ which is larger in settling rate than impurities can be increased, improving the purity of $CaCO_3$ and decreasing the water content so much. But the capture of $CaCO_3$ gets lowered.

2) By increasing the bowl speed, settling rate of solids in the feed sludge toward bowl and the density of sludge deposited on the bowl can be increased. But the relative velocity of $CaCO_3$ to impurities remains unchanged. Accordingly, the capture of $CaCO_3$ increases, and the water content in the dewatered sludge decreases. But the purity of $CaCO_3$ remains unchanged.

3) Pool depth corresponds to the scraping-out depth of dewatered sludge. In the tests, if there are strata as shown in Fig. 5.2, it may be theoretically agreed that the water content of dewatered sludge can be decreased, and the ratio of $CaCO_3$ in dewatered sludge can be increased without changing the capture of $CaCO_3$ largely if the pool depth is adjusted to the depth falling just on the boundary surface between the middle and bottom

layers shown in Fig. 5.2. It is inferred that in the tests the pool depth was located above the boundary surface between the middle and bottom layers.

4) By increasing the conveyor speed, the detention time of sludge deposited on the bowl is shortened as sludge is discharged quickly one after another. Eventually, therefore, the pool depth is considered likely to become larger. Hence, it is concluded that to increase the conveyor speed is to increase the pool depth.

To generalize the results within the scope of the tests, the centrifugal force should preferably be set at 3,000 G and the pool depth at No. 1 (0.5 ~ 0.8 mm) and the conveyor speed at 4.7 rpm. in order to obtain dewatered sludge of low water content and with high capture of calcium carbonate of high purity from lime sludge.

The sludge feed rate, when increased, increases the purity of $CaCO_3$ in the dewatered sludge, but decreases the capture of $CaCO_3$ to the contrary. Further investigations are needed in this respect for the purpose of finding out the optimum conditions. The centrate contains a considerably large amount of solids (9,200 mg/lit. to 13,500 mg/lit. in the tests) as part of solids in the thickened sludge is transfered into the centrate. For this reason, the solids in the centrate are required to be dewatered and separated. The results of dewatering test on the centrate by means of the pilot plant centrifuge are shown in Table 5.7. The centrate was hard to dewater; even when the centrifugal force was increased up to 3,000 G, the capture of solids was only 47.3 % with the water content left as high as 84.8%. Thus, the centrate of centrate still contained no less than 6,500 mg/lit. of solids. Namely, sharp increase in the capture of solids is hardly expected unless chemical dose is practised. Use of dewatering aid, such as polymer, or practice of alternative method will be necessary.

5.3.2 RESULTS OF RECARBONATION OF LIME SLUDGE

Precipitation rates of $CaCO_3$, $Mg(OH)_2$, $Ca_5(OH)(PO_4)_2$, which are main components of lime sludge, are increased with rise in pH value, and they are solubilized when pH value declines. If the solubilizing rate of $CaCO_3$ is smaller than that of $Mg(OH)_2$ and $Ca_5(OH)(PO_4)_3$, recarbonization of lime sludge can improve the purity of $CaCO_3$ in the lime sludge, it will provide an effective way for the reuse of lime sludge. Then, the recarbonation of lime sludge was tested at the Yokosuka Pilot Plant, with the results shown in Fig. 5.5

According to Fig. 5.5, it is found that Yokosuka Pilot Plant's lime sludge used for the tests has smaller solubilizing rate of $CaCO_3$ with reduction in pH than $Mg(OH)_2$'s and $Ca_5(OH)(PO_4)_3$'s, and therefore that it is feasible to recarbonate the lime sludge. By recarbonating feed lime sludge from pH 11.29 to pH 8.11, $CaCO_3$ was solubilized 16%, and $Mg(OH)_2$ and $Ca_5(OH)(PO_4)_3$ 82% and 30%, respectively.

In that case, the composition in dried sludge changed as shown in Fig. 5.6; while $CaCO_3$ accounted for 56.9% in raw lime sludge, it increased up to 72% after recarbonation. On the other hand, $Mg(OH)_2$ changed from 22.8% to 6.1%.

5.3.3 SUMMARY

In order to effectively recover lime from the lime sludge, it is necessary to recover majority of calcium carbonate, the material of lime, with high purity and with less water content in dewatered sludge, from the lime sludge in a preliminary stage.

As ways to this end, the centrifuge method and the recarbonation method were examined, and the following conclusions are attained.

1) Calcium carbonate contained in lime sludge is larger in grain size and specific gravity than impurities such as magnesium hydroxide and calcium hydroxyapatite, and is excellent in dewaterability.
When lime sludge is put to centrifugal separation process, calcium carbonate is trapped selectively in the bottom layer. By recovering the bottom deposit in the form of dewatered sludge, the above objective is achieved.

2) There are four factors influential to the centrifugal separation. They are sludge feed rate, centrifugal force, pool depth and conveyor vevolution. All these have effects on the purity and capture of calcium carbonate and water content of dewatered sludge.

3) The optimum conditions in this test condition under which to operate the centrifuge for dewatering lime sludge to recover lime are: centrifugal force, 3,000 G; pool depth, No. 1 (0.5 ~ 0.8 mm); conveyor speed, 4.7 rpm. As regards the sludge feed rate, further investigations are required.

4) Solids in the centrate are hard to recover without chemical injection by centrifuge. It is therefore necessary to study alternative methods.

5) As regards the lime sludge used in the tests, recarbonization improves purity of calcium carbonate without degrading its capture.

Table 5.1 Results of Lime Precipitation (Average of 1973 ~ 1974)

Experiment Conditions Lime Dosage 300 mg·$Ca(OH)_2$/ℓ

Overflow Rate of Lime Sedimentation Tank: 33 ~ 50 m^3/m^2·day
Polymer Dosage: 0 ~ 0.2 mg/ℓ
Return Sludge Rate: 0 ~ 20%

	pH	Turbidity (mg/ℓ)	S.S. (mg/ℓ)	P (mg/ℓ)	P, ortho (mg/ℓ)	BOD (mg/ℓ)	$COD_{cr.}$ (mg/ℓ)
Influent (secondary effluent)	7.24	4.6	4.1	1.464	1.265	8.41	31.9
Effluent from lime sedimentation tank	10.53	12.2	19.5	0.326	0.262	5.22	25.4
Removal (%)	–	–	–	77.7	79.3	37.9	20.3

Table 5.2 Results of Lime Sedimentation Tank Operation

Lime Dosage: 300 mg·$Ca(OH)_2/\ell$
Overflow Rate of Lime Sedimentation Tank: $50m^3/m^2 \cdot D$

Condition / Item		Fe^{+++} addition (mg/ℓ)				Return sludge (% to influent flow)				Polymer additon (mg/ℓ)		
		0	0.5	2.0	5.0	0	10	20	30	0	0.1	0.5
Temp. of influent (°C)		18.7	19.5	20.5	19.9	22.0	21.3	21.0	22.2	24.5	24.6	24.8
pH	① *	7.20	7.34	7.24	7.28	7.24	7.29	7.22	7.42	7.46	7.26	7.36
	② **	10.23	10.26	10.31	10.28	10.21	10.24	10.29	10.13		10.35	9.31
SS (mg/ℓ)	①	22.8	24.0	31.0	27.0	9.4	11.6	16.8	37.0	4.3	11.7	25.0
	②	18.5	16.0	10.2	55.2	13.0	23.2	31.6	26.4	21.6	27.0	40.5
P (mg/ℓ)	①	1.487	1.906	3.320	2.054	2.538	1.709	2.170	2.020	1.895	1.864	1.116
	②	0.313	0.417	0.436	0.276	0.472	0.213	0.287	0.345	0.452	0.403	0.182
P-D (mg/ℓ)	①	0.836	1.331	2.265	1.666	1.882	1.226	1.569	1.297	1.155	1.647	0.649
	②	0.151	0.189	0.209	0.120	0.277	0.074	0.094	0.104	0.287	0.163	0.104

* ① Influent (Secondary effluent) ** ② Effluent from lime sedimentation tank

Table 5.3 pH, Phosphorus (P), and Dissolved Phosphorus of Each Unit Process Effluent

	pH	P (mg·P/ℓ)	P-D (mg·P/ℓ)	P-D/P
Influent (secondary effluent)	7.24	1.554	1.511	0.972
Effluent from lime sedimentation tank	10.35	0.514	0.044	0.081
Effluent from ammonia stripping effluent	9.89	0.408	0.181	0.444
Effluent from recarbonation tank	7.47	0.337	0.304	0.902
Effluent from filter	6.92	0.352	0.350	0.994

Table 5.4 Materials for Lime Precipitation Process and Scale Formation

Water temp.: about 14°C
pH: about 10.4

Material / Setting place	Stainless steel	Steel	Rubber	P.V.C.	Planed wood	Wood
Rapid mixing tank	66.1*	68.4	67.8	71.7	62.2	70.2
Lime sedimentation tank	5.2	6.1	6.8	5.9	6.6	1.5
Lime sedimentation tank effluent Holding tank	19.6	22.2	17.1	23.7	18.2	22.1

* Scale weight (mg)/area of test piece (cm^2) after 30 days from test piece setting.

Table 5.5 Results of Scale Formation of Calcium Carbonate Experiment

(Water temp.: about 13°C pH: about 10.4)

Place putted test place	Days passed after test piece putted (D)				
	2	6	8	10	15
10 cm lower from water surface of lime sedimentation tank	0.075	0.316	0.347	0.603	1.489
10 cm lower from water surface of lime sedimentation tank effluent holding tank	0.503	0.645	1.587	4.040	7.695

Scale weight (mg)/area of test piece (cm^2)

Table 5.6 Separation Results of Lime Sludge by Centrifuge for Laboratory

3,000 rpm 10 minutes

		Feed sludge	Dewatered sludge		
			Upper layer	Medium layer	Lower layer
Results of analysis	Ca (mg/g)	255	167	186	313
	Mg (mg/g)	69.2	138	122	41.0
	Fe (mg/g)	3.34	5.58	5.50	1.98
	P (mg/g)	14.4	37.2	26.9	5.17
	VSS (mg/g)	181	284	259	94.4
	Specific gravity	2.250	2.076	2.093	2.276
	Water content (%)	89.5	89.0	82.7	57.4
Rate of each component in dry sludge (%)	$CaCO_3$	60.0	27.1	32.1	75.3
	$Mg(OH)_2$	16.6	33.0	29.2	9.8
	$Fe(OH)_3$	0.6	1.1	1.0	0.4
	$Ca_5(OH)(PO_4)_3$	7.8	20.1	14.5	2.8
	VSS	13.0	18.2	16.9	6.4
	The others	2.0	–	4.3	3.3
Rate of each component captured in each layer (%)	Solids		20.6	14.6	64.8
	$CaCO_3$		9.5	7.9	82.6
	$Mg(OH)_2$		39.0	24.5	36.5
	$Fe(OH)_3$		35.9	23.1	41.0
	$Ca_5(OH)(PO_4)_3$		51.3	26.2	22.5
	VSS		36.2	23.8	40.0
	The others				

Table 5.7 Results of the Second Stage Dewatering

Caracter of the First Stage Centrate

SS: 11,800 mg/ℓ
Water Content: 98.8%
Ca: 105 mg/g·dry Sludge
Mg: 132 mg/g·dry Sludge
Fe: 20.5 mg/g·dry sludge
P: 19.7 mg/g·dry sludge
VSS: 330 mg/g·dry sludge

			1	2	3	4
Dewatering condition	Centrifugal force (G)		1,000	2,000	3,000	4,000
	Sludge feed rate (m^3/h)		0.917	0.798	0.822	0.895
	Conveyer revolution (rpm)		6	6.7	8.1	9.4
	Pool depth		3	3	3	3
Results of dewatering	Water content (%)		81.3	83.3	85.3	84.8
	Rate of each component captured in dewatered sludge (%)	Solids	26.1	37.5	40.5	47.3
		Sludge volume	1.1	2.2	2.7	3.2
		$CaCO_3$	81.0	86.6	84.1	87.6
		$Mg(OH)_2$	36.6	49.7	57.5	59.3
		$Fe(OH)_3$	27.9	42.9	52.1	55.5
		$Ca_5(OH)(PO_4)_3$	32.6	45.2	53.6	55.7
		VSS	42.6	51.5	60.4	63.9
		The others	61.1	66.7	55.6	67.6

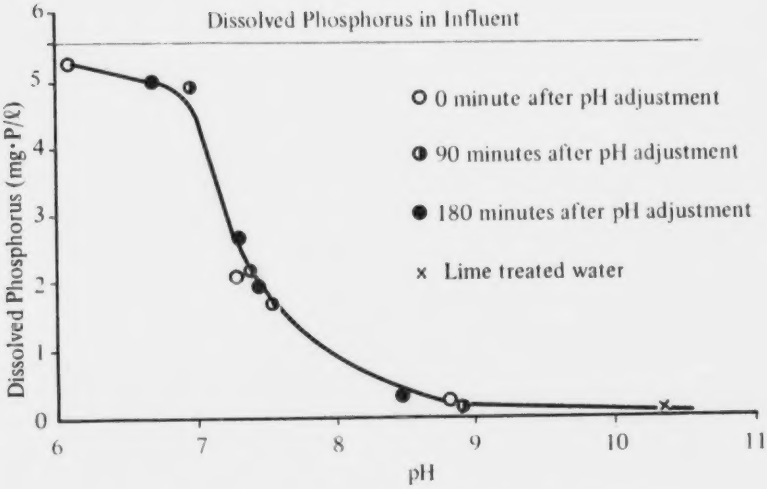


Fig. 5.1 pH Lowering vs. Resolublization of Phosphorus

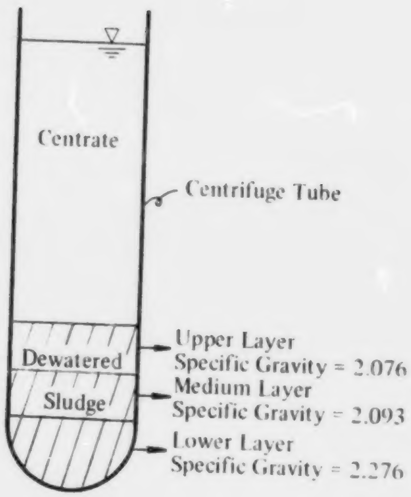

Centrate
Centrifuge Tube
Dewatered
Sludge
Upper Layer
Specific Gravity = 2.076
Medium Layer
Specific Gravity = 2.093
Lower Layer
Specific Gravity = 2.276

Fig. 5.2 Result of Lime Sludge Centrifugal Separation

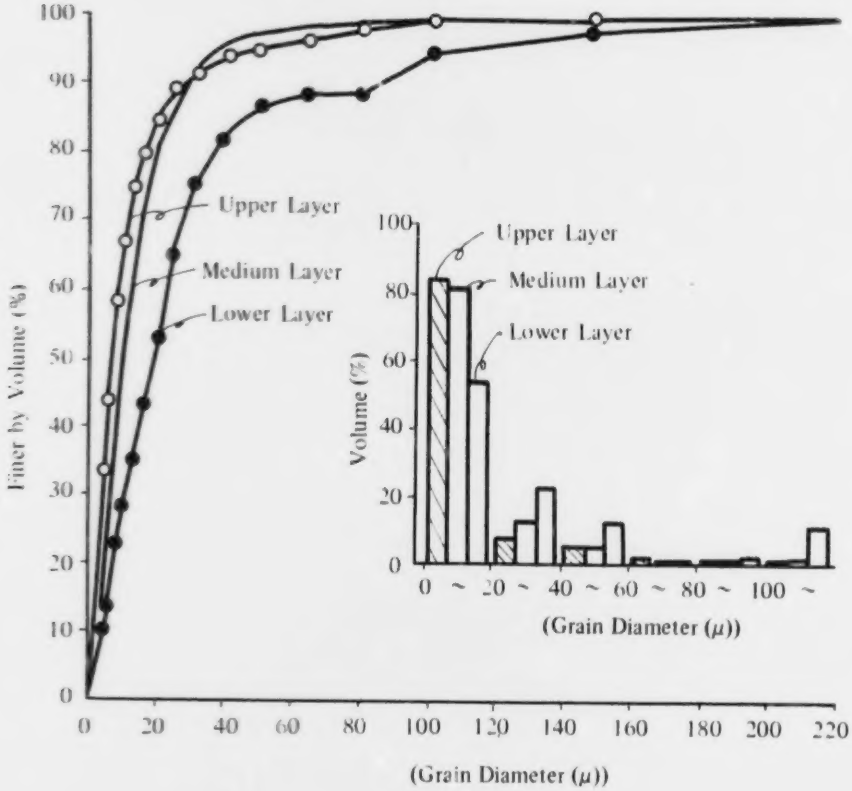


Fig. 5.3 Grain Size Cure of Each Layer

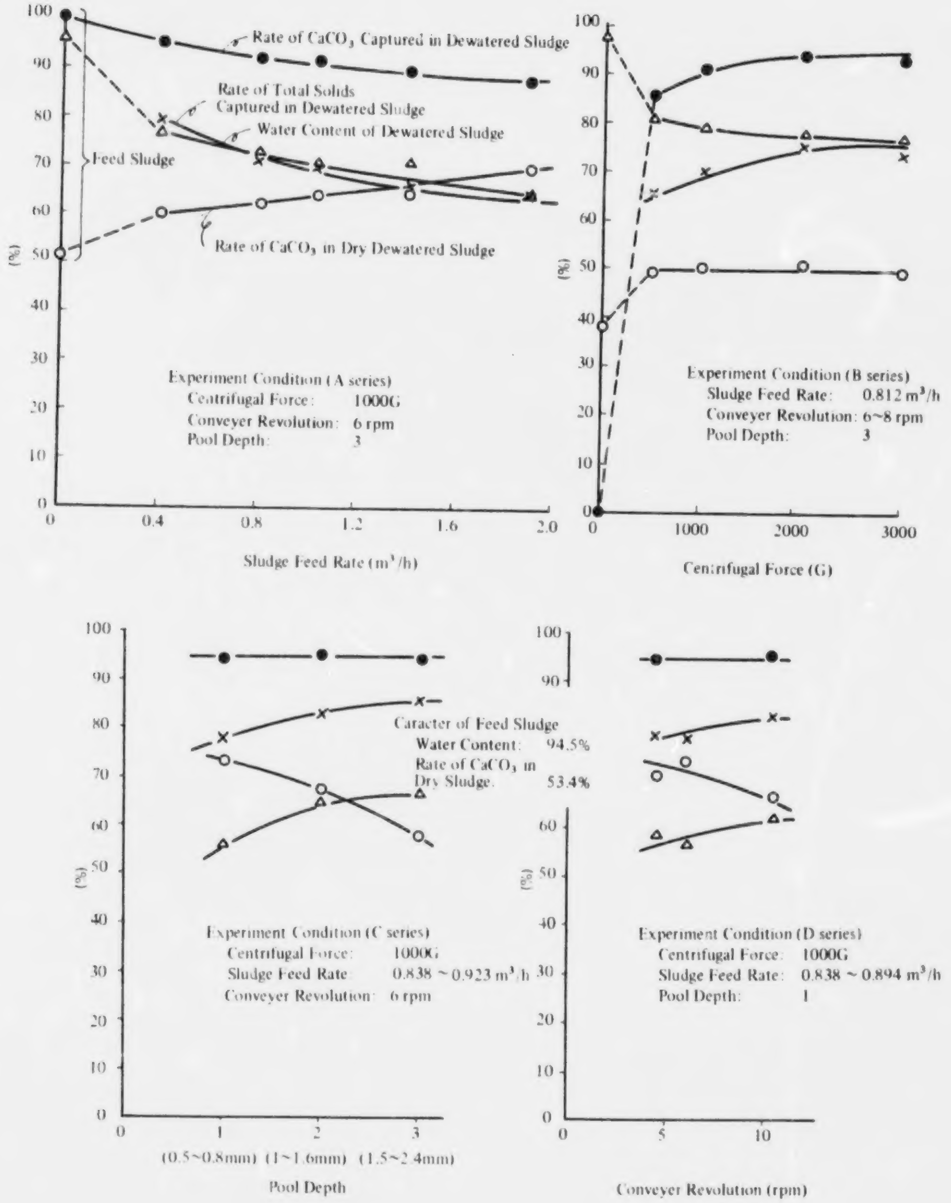


Fig. 5.4 Dewatering Results of Lime Sludge by Centrifuge for Pilot Plant

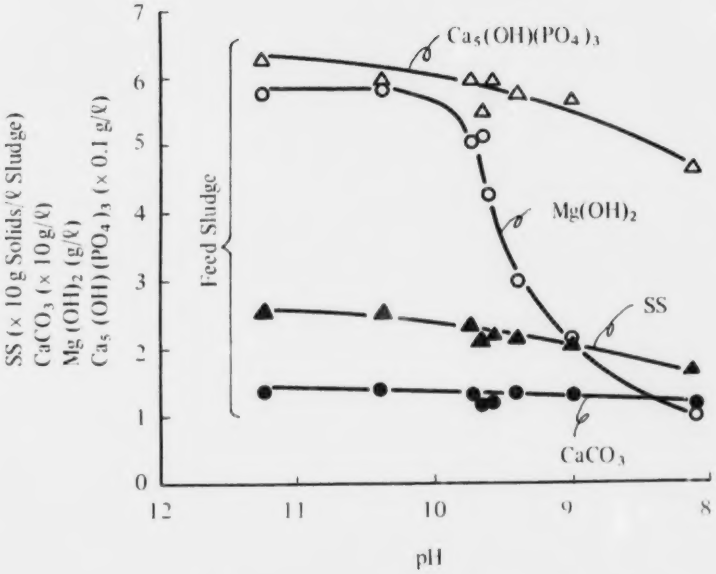


Fig. 5.5 pH Reduction and Each Component Change of Lime Sludge by Recarbonation

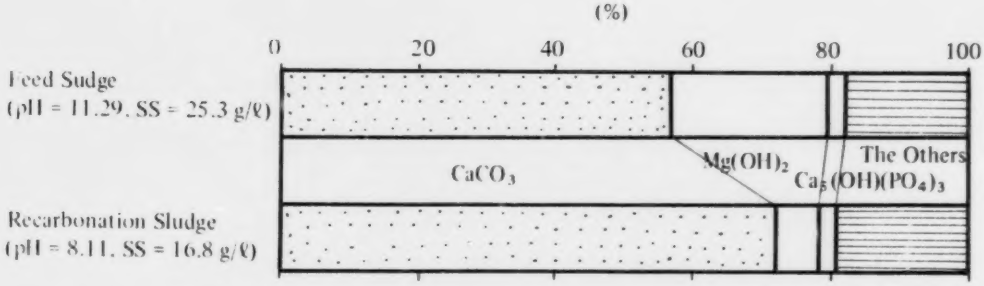


Fig. 5.6 Rate of Component in Feed Sludge and Recarbonation Sludge

CHAPTER 6. ADSORPTION BY ACTIVATED CARBON AND REGENERATION OF SPENT CARBON

6.1 Adsorption Studies by Granular Activated Carbon 228
  6.1.1 Outline of Experiment Facilities 228
  6.1.2 Conditions of Experiment 228
  6.1.3 Results of Adsorption Experiments 228

6.2 Regeneration of Granular Activated Carbon 232
  6.2.1 Introduction 232
  6.2.2 Reactivation by Multi-Hearth Furnace 232
  6.2.3 Arrangement for Experiment 232
  6.2.4 Experiments 234
  6.2.5 Measuring Items and Methods of Measurement 234
  6.2.6 Regenerating Conditions 235
  6.2.7 Results of Regeneration 235
  6.2.8 Summary 237

6.3 Laboratory Regeneration Test of Granular Activated Carbon 239
  6.3.1 Introduction 239
  6.3.2 Laboratory Equipment Used 239
  6.3.3 Testing Method 239
  6.3.4 Results of Tests 240

6. ADSORPTION BY ACTIVATED CARBON AND REGENERATION OF SPENT CARBON

6.1 ADSORPTION STUDIES BY GRANULAR ACTIVATED CARBON

6.1.1 OUTLINE OF EXPERIMENT FACILITIES

Kyoto Pilot Plant is equipped with six contact column using granular activated carbon. They are paired off into three groups, permitting three tests at the same time.

Flow diagram of the process is as illustrated in Fig. 6.1. The dimensions and other particulars are as follows.

Dimensions:	surface area, 0.76 m × 0.93 m = 0.7 m^2 allowable maximum head loss, 5.0 m
Media support:	Leopold block, gravel layer (25 cm)
Washing:	back washing and surface washing in combination
Processing capacity:	maximum overflow rate, 600 $m^3/m^2/d$. (17.5 m^3/hr/column)

The characteristics of activated carbon charged into the column are as listed in Table 6.1.1.

The colum are so pipe harnessed as to permit other three running modes single-column 6-group, 3-column 2-group, and 6-column 1-group.

6.1.2 CONDITIONS OF EXPERIMENT

In the adsorption experiment, column have been operated for some 12 months or around 35,000 m^3 in terms of the volume of wastewater passed through per 0.7 m^2 after being charged with fresh carbon and for some 6 months or 13,000 m^3 in the volume of wastewater after regeneration of carbon.

As regards the fresh carbon, LV 10 m/hr has been taken as standard. As regards the regenerated carbon, the contact time has been fixed at 18 min. per column, and LV has been set at around 9.4 m/hr, a little lower than fresh carbon, to meet the decrease of bed depth due to regeneration loss. Wastewater flow in the column is downward.

For the adsorption experiment the filtered secondary effluent obtained from the Toba Sewage Treatment Plant has been used. In filtration, $Al_2(SO_4)_3$ has occasionally been dosed at a rate of about 0.65 – 1.96 mg/l as Al. There has been practised no particular flocculation sedimentation to speak of, so majority influent for the experiment may be said to be simply filtered secondary effluent.

The influent flow down through the column gravitationally, and its SS gradually blocks up carbon bed, decreasing actual LV.

In order to avoid this, the primary column has been put to back washing about once every three days. Backwash flow rate has been set at 0.74 to 0.9 $m^3/m^2/min$, though different slightly dependent on the columns.

6.1.3 RESULTS OF ADSORPTION EXPERIMENTS

The results of experiments so far conducted show that the adsorption effect is slightly different with the kind of carbon. And the carbons have much in common from the view point of time-dependent patterns.

The discussion here is therefore referred to Brand-A (8 x 30) for the secular change and to the data obtained in 1975 (regenerated carbon) for the differences due to difference in quality of carbons.

(1) Removal of organic matter

Secular changes of indices for organic matter – BOD_5, COD_{Mn} and TOC – are shown in Figs. 6.2 through 6.4. As is clear from these figures, the organic matter removal capacity of activated carbon is not so high rather than which we expected.

BOD_5 : –

Removal of ordinary BOD_5 by fresh carbon is small. Worse, it sometimes runs by contraries; there are many such cases as BOD_5 becomes the worse by the adsorption treatment with activated carbon.

In many cases, the effluent of the secondary column has larger BOD_5 value than that of the primary column. In the case of regenerated carbon, the removal is about 75% at the beginning of restart. After one month operation however, it declines to 30%.

As regards the fresh carbon, BOD_5 value is larger in the effluent of secondary column than in the effluent of the primary column, all the way from the beginning of operation.

This rise of BOD_5 value in the effluent of the secondary column was suspected to be attributable to the promotion of nitrification in the bed of activated carbon, and studies were made in this respect accordingly.

Fig. 6.5 shows BOD_5 and forms of nitrogen in influent and contactor column effluents, which are based on the data obtained in May 1975.

On May 8, BOD_5 in the effluent of the secondary column was smaller than in the effluent of the primary column. On May 29, the situation was reversed. BOD_5 were measured at once in ordinary method and in a special method in which nitrifying bacterias were suppressed. BOD_5 by the former method is named T-BOD_5, and that by the latter C-BOD_5. From Fig. 6.5, the following are noticed.

(A) The larger the change of NO_3-N, the more reasonable the decrease of T-BOD_5. If the change of NO_3-N is small, T-BOD_5 is inverted.

(B) Even when T-BOD_5 is inverted, C-BOD_5 decreases reasonably.

In additions to these facts, decreae of DO is also noticed. Also, various bacteris are identified in the activated carbon contactor column. Hence, inhabitation of nitrifying bacteria in the column permits of no doubt. It is therefore judged that the increase of T-BOD_5 is due to nitrification. In support of this, the pilot studies conducted by the Bureau of Water Works, Tokyo Metropolitan Government, for the production of industrial water from wastewater revealed lots of nitrite-forming bacteria and nitrate-forming bacteria present in the effluent of activated carbon contactor column.

In view of these facts, use of T-BOD_5 as a characteristic to provide a guideline for activated carbon treatment will be disagreed. In order to remove T-BOD_5 which has been taken up as an index for organic matter in the Environmental Water Quality Standard and the Effluent Limitations Guidelines in Japan, the treatment using with activated carbon should be reviewed.

COD_{Mn} : –

Both fresh carbon and regenerated carbon show small removal of COD_{Mn} contrary to our expectation. Even at the beginning of operation, fresh carbon remained at the level of 30% in removal with a contact time of 18 min. (primary column effluent) and about 70% for a contact time of 36 min. These values were reduced to 25% and 50% respectively in 1 to 2 months after start-up of operation. The removal thereafter decreased gradually to some 30% even with 36 min. of contact.

This tendency remained the same after regeneration: Only for 2 to 3 months, 50% or more of removal could be attained stably. In the experiments, there was no dramatic drop in COD_{Mn} removal, and it could not be found whether break-through was developed or not.

No direct measurement of COD_{Mn} adsorption capacity was made as to fresh carbon, but the following was inferred to be according to TOC isothermal adsorption before and after activated carbon treatment.

(i) From Fig. 6.6, the following equation is derived.

$$\text{TOC (mg/}\ell) = 0.889 \times \text{COD (mg/}\ell) + 1.748 \qquad (1)$$

Taking the average effluent TOC as 10 mg/l.
as suggested by Fig. 6.4, we obtain COD = 93 mg/l. from Eq. (1).
Namely, at a TOC concentration of around 10 mg/l.,

$$\frac{COD_{Mn}}{TOC} = \frac{9.3}{10} = 0.93 \qquad (2)$$

(ii) TOC Freundrich line of fresh carbon in Fig. 6.14 is given by the following formula.

$$x/M = K.C^{1/n} = 0.006.C^{1.45} \qquad (3)$$

Hence, at TOC 10 mg/l. (This value is over the chart-range), x/M ß 0.169 mg/mg ß 169 kg/t.
From Eq. (2), this is converted into COD as follows.
169 × 0.93 = 157.2 kg/t

(iii) On the other hand, the total, weight of adsorbed matter, L, can be expressed by the flow rate, Q, and COD_{Mn} concentration difference between influent and effluent of carbon column $(Ci - Co)$ as follows.

$$L = \Sigma\ (Ci - Co).Q.\Delta t$$

Cumulative total volume of water, V, is given by the following formula.

$$V = \Sigma Q \Delta t$$

Thus, Fig. 6.7 is obtained.

(iv) With reference to Fig. 6.6, after treating 34000 m^3 of filtered secondary effluent, fresh carbon removed only 105 kg COD_{Mn} in the primary column and 90 kg COD_{Mn} in the secondary column. On the other hand, the adsorption capacity per column is calculated from (ii) as follows.
157.2 kg/t × 0.7 m^2 × 3 m × 0.5 t/m^3 = 165.1 kg. Namely, the removal is 65%

in the primary and 55% in the secondary.

It is therefore considered that this denies the breakthrough of the activated carbon.

(v) In addition, by backwashing the column, a great quantity of suspended solids and COD is detected in the backwash wastewater as shown in Figs. 6.8 and 6.9, showing that the activated carbon bed serves not only as an adsorber but as a filter. For example, the four measurements conducted in the summer of 1974 are as shown in Table 6.2; removal of COD_{Mn} in the primary column by filtration was 0.034 ~ 0.125 kg per 168 m^3 of water (0.2 ~ 0.74 g/m^3)

(vi) Taking an average COD_{Mn} removal of 0.43 g/m^3, 34,000 m^3 releases 14.6 kg or 14% of removal by filtration. Namely, the weight ratio of actually adsorbed COD_{Mn} to the capacity is no more than 50% even in the primary column.

For all that the breakthrough is theoretically denied as above, why COD_{Mn} removal is extremely low.

The following may be suspected to be.

Ⓐ COD_{Mn} lacks eligibility for organic matter index.

Ⓑ Influent is anomalous in quality.

As regards Ⓐ, however, TOC removal is much of a muchness, and should be deleted. As regards Ⓑ, the influent used for the experiments was filtered secondary effluent of activated sludge process.

The influent applied to the activated carbon adsorption process which usually raises 90% or more efficiency is always put following to chemical coagulation, and effect of the omission of this coagulation process may be a cause. To corroborate this, we are going to proceed a laboratory experiment. From 1976 on, this will also be corroborated on a pilot plant scale.

TOC:

Like COD_{Mn}, TOC removal is also low and far from our expectation. In the case of fresh carbon, the removal is below 50% inclusive of the secondary column from the beginning of operation, which is worse than COD_{Mn} removal.

In the case of regenerated carbon, the removal is more than 50% at the beginning with 36 min. of contact, but is still far less on the whole. Because we lack satisfactory data we can not make any analysis like as Fig. 6.7. Just as with COD_{Mn}, however, the breakthrough is unlikely.

This low removal will be ascribable to the peculiarities of the influent employed.

(2) ABS removal

Application of activated carbon to water and wastewater treatment is to remove organic matter, deodor, remove colouring matter and also to remove ABS.

The pilot plant experiment discussed here placed emphasis to the removal of organic matter, but also handled ABS removal to some extent. Fig. 6.10 shows an example.

In the case of fresh carbon, you can find lower removal at the beginning of operation, fourth month and eight month. But ABS removal is higher and more stable. Comparing with the removal of organic matter. ABS removal recorded is around 50% with 18 min. of contact and around 80% with 36 min. of contact. In the case of regenerated carbon, the removal is even more stable; even after 6 months from start, the removal remained 90% with 36 min. of contact. But, the removal with 18 min. of contact plunges to 40 to 50% from about 5th month, and the breakthrough is suspected to be.

ABS removal is surprisingly high compared with other items, and this is not limited to this brand (Brand A), but is seen common to other brands like as Fig. 6.11. The figure shows the results of measurement made 5 and odd months after start-up operation using regenerated carbon.

6.2 REGENERATION OF GRANULAR ACTIVATED CARBON

6.2.1 INTRODUCTION

The regeneration is carried out by heating, chemical extraction, oxidation and decomposition and biodegradation and so forth, depending on the kind of activated carbon, adsorption conditions and matter to be adsorbed. If liquid-phase adsorption, like as in sewage treatment is practised, the surface of activated carbon particles are foued up inversibly and it becomes to be difficult to regenerate.

For this reason, the thermal regeneration (baking reactivation process) is usually put to practice. Dealt with here are the findings from the experiment on reactivation of spent activated carbon by multi-hearth furnace.

6.2.2 REACTIVATION BY MULTI-HEARTH FURNACE

The following are the conditions required for the regenerator of spent activated carbon.

(1) High thermal efficiency

(2) Easy control of zone temperature and atmosphere in drying, baking and activating processes

(3) Less burning-off in baking and activating stages

(4) Exposing the surfaces of activated carbon particles with high frequency

(5) Easy control of detention time to meet degree of adsorbed weight of matter

(6) Less susceptibility to mechanical failure and wear loss

(7) High capability of covering a wide range of processing rates

(8) Easy operation and maintenance

(9) Established and proven structure as equipment

The multi-hearth furnace employed for the experiment is qualified with respect to the above requirement, and has widely been used.

6.2.3. ARRANGEMENT FOR EXPERIMENT

(1) Specifications of principal equipment

The experiment installation is composed of a furnace, spent carbon feed system, regenerated carbon withdrawal system, exhaust gas treatment system and a steam generator.

(i) Furnace

Type : 6-hearths type
Dimensions : 750 φ × 1,500 H
Shaft drive : 0.23 – 2.3 rpm, variable, 0.4 kW
Main materials : Steel + refractory
Ancillary equipment : Gas burner × 8 units

(ii) Steam generator (packaged boiler)

Type : Low-Pressure type
Capacity : 0.[illegible] kg/cm^2, 75 kg/hr
Quantity : 1 unit

(2) Arrangement flow for Experiment as per Fig. 6.12.

(i) Spent carbon feed system

Spent carbon is withdrawed from the contact colum in the form of slurry, put into a dewatering tank where it is dewatered roughly, and then is charged by the screw conveyor onto the lst hearth of the furnace at a constant rate. Here the moisture content of the spent carbon is about 45 to 50%.

(ii) Regeneration method

The spent carbon thus fed, which has a water content of 45 to 50% is heated at 250 to 950°C in the furnace, and is reactivated after passing through drying, baking and activating processes.

The furnace is equipped with six hearths and its 2nd, 4th, 5th and 6th stages are each equipped with two low-pressure velocity gas burners using propane gas as a fuel.

In order to promote the reactivation, the 4th, 5th and 6th stage are each equipped with two nozzles supplying steam as an oxidizing gas.

(iii) Regenerated carbon withdrawal system

The regenerated carbon is then discharged from the withdrawal chute equipped on the 6th stage hearth into a quench tank without being exposed to the open air and quenched. The quench tank is always supplied with water to keep its temperature below a specified level. Then, the regenerated carbon is withdrawed from the quench tank in the form of slurry and dewatered in a dewatering tank.

(iv) Exhaust gas treatment system

The exhaust gas from the furnace contains stinky gas volatilizing at a low temperature in the drying process, organic gas developed by decomposition in the baking process and reducing gas from the reactivating process. Also it contains carbon particulates developed by mechanical grinding in the form of dust. In order to completedly burn out stinky gas, decomposed gas, reducting gas and sut carbon, an after-heater is located just behind the furnace.

The after-heater is held at a temperature of 800 to 850°C, and in it the exhaust gas is detained for about 0.5 to 1.0 sec.

The exhaust gas treated in the after-heater is then introduced into a scrubber where it is cooled and moistened to remove dust and oxidizing gas, and then is vented to the open air.

6.2.4 EXPERIMENTS

In order to investigate reactivating condition for multi-hearth furnace and activity recovery rate, some experiments were proceed using the spent carbon from the Kyoto pilot plant, and pilot scale furnace in December 1974.

6.2.5 MEASURING ITEMS AND METHODS OF MEASUREMENT

(1) Measuring items

Three brands of activated carbon, A, B and C, are to be measured as to the following items.

(i) General characteristics

Apparent density, Particle size Hardness number, Fixed residue, Methylene blue number, Iodine number, Molasses decolorizing index, Phenol value, ABS value

(ii) Adsorption capacity for organic matter in water

TOC Freundrich isotherms using filtered secondary effluent

(iii) Micropore characteristics

Surface area of micropore per unit carbon weight, mean micropore size, micropore volume, pore size distribution

(iv) Weight of adsorbed matter

(2) Outline of measuring methods especially used in Japan

(i) General characteristics

Phenol value:

According to JWWAK 113 (Japan Water Works Association Standard: Measuring Method of Powder Carbon for Water Works)

100 ppb phenol solution is taken as a control, and the amount of activated carbon (ppm) required to reduce it to a concentration of 10 ppb is measured. This amount is called phenol value.

ABS value:

According to JWWAK 113.

The amount of activated carbon (ppm) required to reduce the concentration of an ABS (sodium alkyl benzene sulphurnate) solution from 5 ppm to 0.5 ppm is called ABS value.

(ii) Adsorption capacity for organic matter in water

Filtered secondary effluent was used for measurement sampling from the Kyoto Pilot Plant on Dec. 23 and 25, 1974. 0.3% and 0.08% powder carbon suspensions were prepared, and 1.0 ml and 0.5 ml portions were taken respectively and added to 20 ml of control solutions. The mixtures were shaken for 2 hrs and filtered.

The filtrate was acidified with hydrochloric acid, and dissolved carbon dioxide was removed by bubbling. Then, TC was measured with a TOC analyzer.

Change of liquid volume due to addition of suspension was corrected, and Freundrich equation was applied to adsorbed weight and residual TOC concentration for refining the data.

(iii) Weight ratio of adsorbed matter to fresh carbon

Dry sample of spent carbon was taken 100 g into an electric furnace using nitrogen gas as atmosphere gas and controlled at 900°C, and was heated for 1.5 hrs. From the resultant weight reduction, the weight rate of adsorbed matter to fresh carbon was calculated using the following formula.

$$\text{Weight ratio of adsorbed matter to fresh carbon} = \frac{\text{Reduction at } 900^\circ\text{C}}{\text{Yield at } 900^\circ\text{C}}$$

Components to be deposited and occluded into carbon without volatilization during the heat treatment at 900°C for 90 min. were neglected.

6.2.6 REGENERATING CONDITIONS

The following are important factors for the reactivation of spent carbon in the furnace.

(1) Hearth temperature and its distribution

(2) Detention time of carbon in the furnace

(3) Hearth atmosphere

(4) Feed rate of activating steam and its position of feed

(5) Loading rate of carbon

In the test, the maximum hearth temperature was set at 930°C, and the activating hearths (4th, 5th and 6th stage) were all set at the same temperature.

The detention time was set at 30 min. irrespective of the kind and feed rate of spent carbon. In order to avoid burning off due to air leakage, the furnace atmosphere was controlled at +2 to +5 mm H_2O throughout the furnace. O_2% in each stage was controlled below 0.1% using O_2 monitoring meter under any specific conditions. As regards the feed rate and kind of carbon, the activating stage temperature was changed with the apparent density as a basis which could provide the fastest mean of judging the recovery rate.

The steam feed was set at 0.6 to 0.9 kg/kg RC; the fourth stage was supplied some one third and the sixth stage some two thirds. The loading rate was set at 25 to 35 kg/m^2 hr. based on wet weight of spent carbon. The operating conditions in regeneration for these spent carbon and consumption rate of fuel aid, etc. are shown in Table 6.3.

6.2.7 RESULTS OF REGENERATION

(1) Recovery rate

The recovery rate of spent carbon can be expressed in two ways: one is gravimetric and the other volumetric. The problem involved in the gravimetric recovery rate is that the reduction of carbon size due to mechanical and thermal effects cannot be taken into account in the evaluation of recovery rate.

Another problem is the inorganic matter which is left after regeneration. The recovery rate should be the gravimetric one determined by the measurement of weight before and after regeneration.

The volumetric recovery rate (A) is shown in Table 6.4. The gravimetric recovery rate is shown in Table 6.5. The gravimetric recovery rate might possibly include errors as water content and weight ratio of adsorbed matter to fresh carbon for calculation were taken at random. By way of reference, volumetric recovery rate (B) as calculated according to another method is shown in Table 6.6. The test results showed as high a recovery rate as from 94 to 98%.

(2) Changes in general characteristics

The measurements of general characteristics of fresh, spent and regenerated carbon are shown in Fig. 6.13. The adsorption capacity of spent carbon was degraded seriously, particularly in the primary column. Table 6.7 shows the results of heat treatment at 900°C in a nitrogen gas atmosphere. It is evident from Table 6.7 that the weight ratio of adsorbed matter to fresh carbon in the primary column is large.

The recovery rate given in Fig. 6.13 denotes the value showing the extent of adsorption capacity with that of fresh carbon taken as 100. It is calculated according to the following formula.

$$\text{Recovery rate (\%)} = \frac{\text{adsorption capacity of regenerated carbon}}{\text{adsorption capacity of fresh carbon}} \times 100$$

Here we define the term "adsorption rate" as the weight ratio of adsorbed matter to the adsorption capacity of fresh carbon. The adsorption rate on each items obtained from Fig. 6.13 is shown in Table 6.8.

As is clear from Table 6.8, ABS value is most largest, followed by molasses decoloring index, and methylene blue number. Usually, the higher the adsorption rate, the worse the recovery. This is corroborated by the experiment. The apparent density is reciprocal to the recovery rate. It is therefore conjectured that the increase of the recovery rate will reduce the recovery rate of apparent density.

(3) Capacity to adsorb organic matter in water

TOC adsorption capacity was measured using filtered secondary effluent. The results were adjusted by Freundrich equation as shown in Figs. from 6.14 to 6.16. 1/n and K in Freundrich equation are shown in Table 6.4. As regards 7 mg/ℓ and 3 mg/ℓ of residual TOC concentration, the recovery of adsorption capacity was from 73 to 110%.

(4) Micropore characteristics

As to various brands, nitrogen gas adsorption curves were determined according to a modified Constone-Inkley method to calculate pore size distribution, and get a relationship between 10 ~ 300Å micropores and their accumulative volume. Also, the relationship between 300Å ~ 15μ micropores and accumulative volume of micropores was determined using the mercury penetration method.

Surface area of micropore per unit carbon weight, mean micropore size, and pore volume of brands are shown in Table 6.10. According to the relationship between accumulative micropore volume and pore diameter, the micropore was graphically differentiated. Pore size distribution is shown in Figs. from 6.17 to 6.19.

The recovery rate of on surface area of micropore per unit carbon weight was 88 ~ 92%, and mean micropore size was increased 2 ~ 9% after regeneration. The recovery rate of total volume of micropores (0 ~ 15μ) was 96 to 103%. Volume of micropores of 300Å ~ 15μ was recovered 101 to 118%, and that for 0 ~ 300Å was reduced. In the micropores of 0 ~ 300Å, 30 ~ 60Å micropore volume was recovered 102 ~ 114%, while 12 ~ 30Å micropore volume and 12Å micropore volume were recovered 80 ~ 90% and 87 ~ 92%, respectively. In order to assess the size of matter adsorbed to the activated carbon from Table 6.10 and Fig. 6.17 ~ 6.19 pore size-wise volumetric adsorption rates are shown in Table 6.11. Adsorbed matter accounted for 68.4 ~ 82% volumetrically of micropores of not exceeding 30Å.

In the regeneration experiment, micropores were reduced both relatively and absolutely, and transitional pores and macropores were increased. It is said that iodine number becomes proportional to the surface area of micropores of larger than 10Å, methylene blue number to that of more than 15Å, and molasses decoloring index to that of more than 28Å. With reference to Fig. 6.13, this is verified as the recovery rate of molasses decoloring index is higher than that of methylene blue number and iodine number.

(5) Results of exhaust gas measurement (Reference)

Measurements of dust, SO_x, NO_x and odorous index, etc. at the outlet of scrubber of after-heating room are shown in Table 6.11.

The dry gas flow rate at the outlet of scrubber was 373 Nm^3/hr which might include a measuring error. Its design value is near actual flow rate of the effluent gas from after-heating room. Odorous index of gas at the outlet of scrubber is seen increased, which might have been due to stripping of organic compounds contained in wastewater because secondary effluent was used for scrubbing. Removal of hazardous materials, except NO_x was high, justifying the performance of the equipment.

6.2.8 SUMMARY

(1) Regeneration achievements by characteristics

Apparatus density:–

As a merkmal for the regenerative recovery rate, the apparent density which can easily be assessed on the spot in a short time was employed, and the recovery rate became around 100.

Mean particle size:–

Brand A reduced by 3.7%. Other brands increased to the contrary. This might have been attributable to inorganic matter and deposit carbon. The increment of ash content was 0.9% for brand A, and 1.2 to 1.9% for brands B and C.

Harness number:–

No remarkable reduction of hardness number of regenerated carbon was noticed.

Methylene blue number, iodine number, molasses decoloring index, phenol value and ABS value:–

The adsorption rate was highest in ABS value, followed by methylene blue number, and molasses decolouring index. The recovery rate was lowest in methylene blue number, followed by ABS value and iodine number. The molasses decoloring index recorded the highest recovery rate, showing 95 to 103%. Iodine number is proportional to surface area of micropores of 10Å and larger and methylene blue number is proportional to that of 15Å and larger. Generally, the higher the adsorption rate, the lower the recovery rate.

But the recovery rate of molasses decoloring index which showed a high adsorption rate is high, and the recovery of transitional pore is easy.

Micropore characteristics:–

From 23 to 35% of the total micropore volume of activated carbon was occupied with adsorbed matter. Of the total adsorbed matter, 68 to 84% went to micropores of 30Å or smaller. Like as general adsorption characteristics, adsorption rate of micropore was high, and the recovery rate was low, accordingly. The micropores of regenerated carbon were reduced relatively and absolutely, and transitional pores were increased relatively and absolutely.

As regards the macropores, there was little difference between fresh and regenerated carbon.

Yield of regeneration (recovery rate):–

Volumetric recovery rate (A) was 94 to 97%, and gravimetric recovery rate was more than 94%. As the apparent density was taken as a merkmal, the recovery rate became high.

(2) Comprehensive Evaluation of the Experiment

As discussed in the foregoing, the capacity recovery rate of spent carbon was 90 to 95%, and the yield was 94 to 97%. The capacity recovery rate is reciprocal to the yield. It is difficult to judge which we should notice as index capacity recovery rate or yield in regeneration recycle of carbon. So long as the experiment is concerned, both showed satisfactory results. The regeneration temperature was very stable. The oxygen gas concentration in the furnace was as low as less than 0.1% on an O_2 monitor, and burning off of activated carbon was not noticed. The running of the furnace was possible for an extended period under steady state conditions.

under steady state conditions.

Adsorbed matter from the secondary effluent was represented by such as adsorbed to micropores of less than 30Å. Iodine number, methylene blue number and ABS value should be taken as merkmal for regeneration; however, if their recovery rate is set at 100% or more, there is a great possibility of getting the yield reduced. For this reason, adoption of methylene blue number which is proportional to comparatively large pores out of macropores and the apparent density which is proportional to the yield will be practically warrantable for the regeneration purposes.

6.3 LABORATORY REGENERATION TEST OF GRANULAR ACTIVATED CARBON

6.3.1 INTRODUCTION

Along with the pilot plant scale regeneration experiment discussed under 6.2, a laboratory regeneration test of spent carbon obtained from Kyoto Pilot Plant has been conducted using a laboratory furnace. The test is still under way, and submitted here are the results obtained so far.

6.3.2 LABORATORY EQUIPMENT USED

The regenerating furnace used is a part of a multi-hearths type furnace. It is composed of a combustion chamber and a heating chamber as illustrated in Fig. 6.20. The heating chamber is equipped with an agitator which permits ploughing-up of sample. The charging of sample is accomplished from the top cover, and the with drawing is carried out from the outlett equipped in the hearth bed while operating the agitator. The performance of the furnace is as follows.

Max. operating temperature in the bottom of heating chamber: 1,100°C

Effective dimensions of heating chamber: 458 mm $\phi \times$ 191 mm H

The temperature control of the heating chamber is undertaken by two thermometric controllers equipped in the chamber and one thermometric controller equupped in outside of the chamber as well as by combustion burners. Temperatures in the furnace are recorded automatically.

6.3.3 TESTING METHOD

(1) Sample

In the test, spent carbon of brand A obtained from Kyoto Pilot Plant was used. The spent carbon was completely saturated with water, and 5 lit. were sampled and drained for 30 min. In this state the water content was 48 to 51%.

(2) Temperature setting

Atmospheric gas temperature in the top zone of the heating chamber were set at 500, 600, 700, 800, 900, and 1,000°C.

(3) Atmospheric gas for regeneration test

Character of the atmospheric gas in the regenerating furnace was as follows.

CO_2 Content	11.6 ~ 15 V/V %
O_2 Content	0 ~ 1.2 "
CO Content	0 ~ 2.3 "

The measurement was carried out with an Orsat gas analyzer. Steam activation was not carried out.

(4) Items measured

The measured items are as follows.

(i) Apparent density
(ii) Iodine number
(iii) Methylene blue number

All these measurements were in accordance with JIS K 0102.

6.3.4 RESULTS OF TEST

(1) The apparent density vs. temperature relationship is shown in Fig. 6.21. The apparent density of regenerated carbon was almost the same level as that of fresh carbon when an atmospheric gas temperature were from 500 to 800°C but was smaller when an atmospheric gas temperature was above 900°C.

(2) The iodine number vs. temperature relationship is shown in Fig. 6.22
The spent carbon was rejuvenated at 800°C almost to the level of fresh carbon. With increase in temperature above 900°C, iodine number increases toward saturation.

(3) The methylene blue number vs. temperature relation-ship is shown in Fig. 6.23. The spent carbon is rejunenate to the level of fresh carbon at 900°C.

(4) When viewed from the above three indices, the optimum atmospheric gas temperature is found to be 800°C if steam activation is not carried out.

Table 6.1 Contactor Design

Colum	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
Brand	A: CALGON SGL 8/30		B: CALGON CAL 12/40		C: TAKEDA SHIRASAGI	
Name of carbon	FILTRASORB 300		FILTRASORB 400		W 8/32	
Particle size	8 × 30		12 × 40		8 × 30	
Bed depth	3,000 m/m					

Table 6.2 COD_{Mn} Removal by Filtration Effect of Granular Activated Carbon Contactor.

	No. 1			No. 3			No. 5		
	L	A	B	L	A	B	L	A	B
Jun. 13	116.6	0.609	0.746	120.25	0.455	0.541	110.6	0.579	0.748
Jun. 27	119.3	0.170	0.204	113.8	0.130	0.163	117.8	0.126	0.153
Aug. 7	55.75	0.175	0.448	55.25	0.134	0.346	55.75	0.162	0.415
Nov. 28	163	0.376	0.330	168.5	0.389	0.330	170	0.463	0.389
Mean			0.432			0.345			0.426

	No. 2			No. 4			No. 6		
	L	A	B	L	A	B	L	A	B
Jun. 13	116.6	0.154	0.189	120.25	0.118	0.140	110.6	0.155	0.200
Jun. 27	119.3	0.144	0.172	113.8	0.104	0.131	117.8	0.123	0.149
Aug. 7	62	0.160	0.369	55.4	0.116	0.299	63	0.190	0.431
Nov. 28	163	0.256	0.224	168.5	0.185	0.157	170	0.225	0.189
Mean			0.239			0.241			0.302

Note: L = Adsorption Run Length
A = Backwashed COD_{Mn} Loading per One Time (kg)
B = Backwashed COD_{Mn} Loading per Unit Volume of treated Water (mg/m^3)

Table 6.3 Operational Condition of Regenerater

Contactor		No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
Brand of Carbon		Brand A		Brand B		Brand C	
Column		Primary Column	Secordary Column	Primary Column	Secordary Column	Primary Column	Secordary Column
Spent Carbon Total Feed Weight	kg WSC	1592.9	1609.9	1445.4	1553.1	1674.4	1657.1
	kg DSC	960.5	931.6	830.7	852.7	961.4	947.5
Spent Carbon	kg WSC	65.5	59.3	58.4	58.4	62.0	59.5
	kg DSC/Hr	39.5	34.4	33.6	32.1	35.6	34.0
Spent Carbon Average Moisture	wt %	39.7	42.1	42.5	45.1	42.6	42.8
Temperature at each Hearth							
Outlet of Exhaust Gas	°C	260 ~ 300	220 ~ 270	250 ~ 280	240	250 ~ 265	220 ~ 240
Second Hearth	°C	510 ~ 520	410 ~ 420	450 ~ 455	390 ~ 400	450 ~ 460	420 ~ 430
Third Hearth	°C	540 ~ 560	485 ~ 495	500 ~ 510	450 ~ 460	495 ~ 510	510 ~ 520
4th Hearth	°C	920 ~ 930	900 ~ 905	910 ~ 920	880	905 ~ 915	890 ~ 900
5th Hearth	°C	920 ~ 930	900 ~ 905	910 ~ 920	880	905 ~ 915	890 ~ 900
6th Hearth	°C	920 ~ 930	900 ~ 905	910 ~ 920	880	905 ~ 915	890 ~ 900
After Heat Room	°C	850 ~ 870	820 ~ 860	850 ~ 860	830 ~ 840	820 ~ 840	820 ~ 860
Inlet to Scrubber	°C	600 ~ 620	590 ~ 620	620 ~ 640	620	600 ~ 620	580 ~ 630
Outlet of Scrubber	°C	70 ~ 80	50 ~ 60	50	50	50	60 ~ 70
Retention Time	min	30	30	30	30	30	30
LPG for Fuel Aid							
4th Hearth	ℓ/Hr	1383.6	1206.0	1198.6	1091.0	1177.0	1182.1
5th Hearth	ℓ/Hr	818.6	779.4	893.0	753.1	842.8	775.5
6th Hearth	ℓ/Hr	959.2	846.1	772.7	687.6	726.1	623.4
Total	ℓ/Hr	3161.4	2831.5	2864.3	2531.7	2745.1	2581.0
Steam Fed							
4th Hearth	kg/Hr	9.3	11.7	10.9	9.9	10.4	9.7
6th Hearth	kg/Hr	19.8	21.3	20.0	20.3	20.6	19.7
Total	kg/Hr	29.1	33.0	30.9	30.2	31.0	29.4
Air Supplied	m^3/Hr (°C)	(30)	(29)	(32)	(28)	(32)	(30)
	m^3/Hr (°C)	81.35	77.68	76.45	71.35	76.89	71.85
Fuel Consumption	ℓ/Hr	5.45	6.36	6.22	7.07	6.0	6.88
Gas Content	CO % (Hearth)	3.0(6)	3.4(4)	2.4(4)	2.8(6)	2.0(2)	2.8(6)
	CO_2 % (Hearth)	12.2(6)	12.0(4)	12.1(4)	12.2(6)	12.1(2)	12.1(6)
	O_2 % (Hearth)	0.11(1~6)	0.11(1~6)	0.11(1~6)	0.11(1~6)	0.11(1~6)	0.11(1~6)

(Note) WSC = Wet Spent Carbon DSC = Dry Spent Carbon

Table 6.4 Volumetric Recovery Rate (A)

Column No.	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
Bed depth before regeneration mm	2,902	2,897	2,744	2,897	3,024	2,895
Bed volume before regeneration m^3	2.03	2.03	1.92	2.03	2.12	2.03
Bed depth after regeneration mm	2,777	2,723	2,670	2,732	2,905	2,782
Bed volume after regeneration m^3	1.94	1.91	1.87	1.91	2.03	1.95
Volumetric recovery rate %	95.6	94.1	97.4	94.1	95.8	96.1

Table 6.5 Gravimetric Recovery Rate

Column No.	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
Wet weight of spent carbon (A) kg	1,592.9	1,609.9	1,445.4	1,553.1	1,674.4	1,657.1
Mean moisture of spent carbon (B) %	39.7	42.1	42.5	45.1	42.6	42.8
Weight ratio of adsorbed matter to carbon (C) %	0.149	0.114	0.193	0.172	0.188	0.134
Dry weight of spent carbon (D) kg	836.0	836.7	696.7	727.5	809.0	835.9
Wet weight of regenerated carbon kg	1,370.6	1,390.6	1.271	1,286.1	1,422.0	1,424.2
Average moisture of regenerated carbon %	41.3	43.4	43.9	44.5	43.0	43.1
Dry weight of regenerated carbon kg	804.1	786.5	712.7	713.8	811.3	810.9
Gravimetric recovery rate %	96.2	94.0	102.3	98.1	100.2	97.0

$$D = A \cdot (1 - \frac{B}{100}) (\frac{1}{1 + C})$$

Table 6.6 Volumetric Recovery Rate (B)

Column No.	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
Dry weight of spent carbon kg	960.5	931.6	830.7	852.7	961.4	847.5
Apparent density of spent carbon	575	552	565	528	570	555
Volume of spent carbon m^3	1.67	1.69	1.47	1.61	1.69	1.71
Dry weight of regenerated carbon kg	804.1	786.5	712.7	713.8	811.3	810.9
Apparent density of regenerator carbon	485	485	450	450	482	482
Volume of regenerated carbon m^3	1.66	1.62	1.58	1.59	1.68	1.68
Gravitational recovery rate %	99.3	96.1	107.7	98.2	99.8	98.5

Table 6.7 The Result of Heat Treatment in 900°C Nitrogen Gas

Items	Brand	A		B		C	
	Column	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
Bed density, backwashed and drained of spent carbon		575	552	565	528	570	555
Iodine number of spent carbon		0.68	0.73	0.67	0.74	0.67	0.73
In 900°C Nitrogen Gas	Recovery rate after 90 min. treatment	87.0	89.7	83.8	85.3	84.2	88.2
	Weight ratio of adsorbed matter to fresh carbon	0.149	0.114	0.193	0.172	0.188	0.134
After heat treatment	Recovery rate (%)	94	96	92	96	102	99
	Apparent density	512	500	486	467	485	500
Iodine number after heat treatment		0.93	0.95	0.96	0.99	0.93	0.95

Table 6.8 Adsorption Rate to the Capacities

Column		No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
Apparent density	%	19.5	14.8	25.8	17.6	14.9	11.9
Methylene blue number	%	52.9	29.4	60.0	35.0	55.6	33.3
Iodine number	%	28.4	23.2	33.0	26.0	29.5	23.2
Molasses decolorizing index	%	55.1	44.3	57.0	41.8	53.5	39.8
Phenol value	%	34.2	–	39.5	3.7	39.5	–
ABS value	%	62.5	–	70.0	–	68.1	–

Table 6.9 Constants in Freundrich Equation and Recovery Rates

Brand		Brand A	Brand B	Brand C
Fresh carbon	1/n	1.45	1.4	1.4
	K	0.006	0.006	0.007
Spent carbon*	1/n	1.75	1.75	1.7
	K	0.0005	0.0004	0.0005
Regenerated carbon	1/n	1.0	1.1	1.05
	K	0.011	0.010	0.010
Comparison of adsorption capacity at 7 mg/ℓ TOC level	Fresh carbon	0.094	0.100	0.099
	Regenerated carbon	0.072	0.084	0.073
	Recovery rate	76	84	73
Comparison of adsorption capacity at 3 mg/ℓ TOC level	Fresh carbon	0.028	0.029	0.030
	Regenerated carbon	0.031	0.033	0.030
	Recovery rate	111	114	100

* The spent carbons are from Primary column

Table 6.10 Micropore Characteristic of Each Carbons

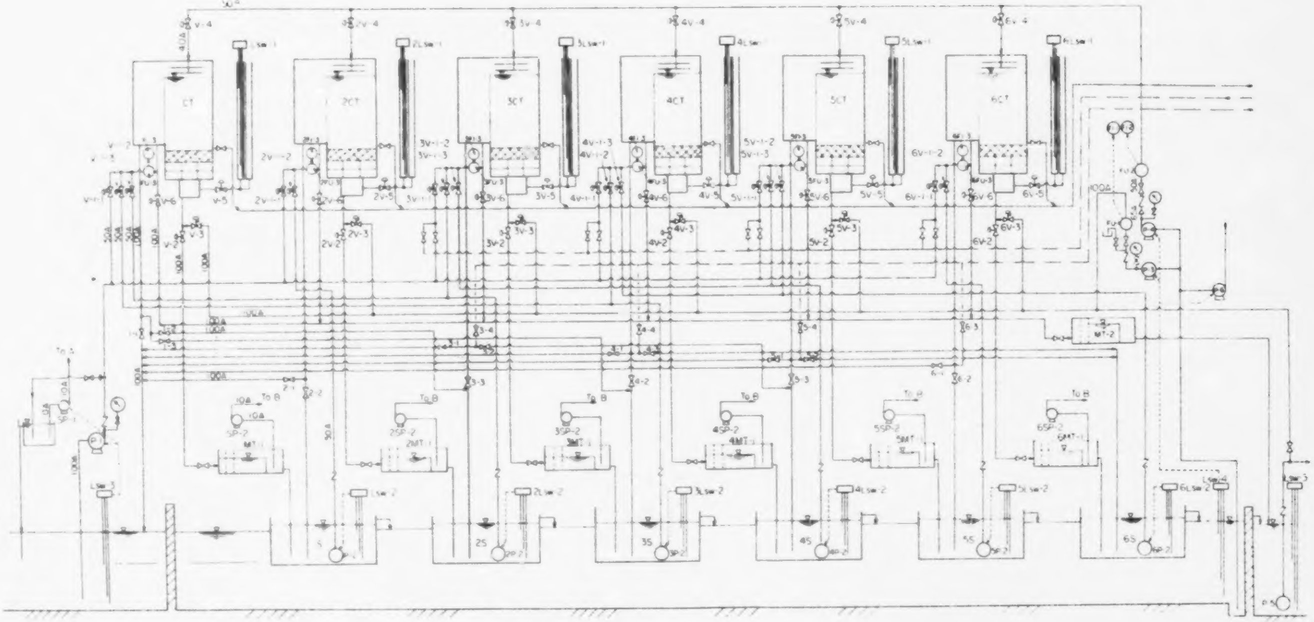
	Brand A					Brand B					Brand C				
	Fresh Carbon	Spent Carbon		Regenerated Carbon		Fresh Carbon	Spent Carbon		Regenerated Carbon		Fresh Carbon	Spent Carbon		Regenerated Carbon	
		No. 1	No. 2		Recovery Rate		No. 3	No. 4		Recovery Rate		No. 5	No. 6		Recovery Rate
Surface Area m^2/g	980	495	610	905	92	1065	470	590	935	88	970	500	580	850	88
Mean Diameter of Micropore Å	20.5	22.5	22.5	22.4	–	21.8	24.0	22.0	22.4	–	20.6	22.0	22.1	22.0	–
Volume of Micropore cc/g 0 ~ 15μ	0.820	0.535	0.629	0.825	101	0.901	0.553	0.610	0.866	96	0.787	0.537	0.594	0.808	103
Volume of Micropore 300Å~15μ	0.317	0.257	0.285	0.319	101	0.321	0.271	0.282	0.343	107	0.287	0.263	0.274	0.340	118
Volume of Micropore ~ 300Å	0.503	0.278	0.344	0.506	101	0.580	0.282	0.328	0.523	90	0.500	0.274	0.320	0.468	89
Volume of Micropore 30 ~ 60Å	0.058	0.039	0.044	0.063	108	0.065	0.043	0.043	0.074	114	0.054	0.037	0.040	0.055	102
Volume of Micropore 12 ~ 30Å	0.176	0.082	0.111	0.163	93	0.218	0.092	0.109	0.177	81	0.163	0.088	0.107	0.131	80
Volume of Micropore ~ 12Å	0.205	0.104	0.119	0.185	90	0.206	0.080	0.111	0.180	87	0.216	0.086	0.126	0.198	92

Table 6.11 Distribution of Absorbed Matter along with Pore Size

Column	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
Total micropores volume 0 ~ 15μ cc/g	0.535	0.629	0.553	0.610	0.537	0.594
Reduction of micropores volume % 0 ~ 15μ	34.8	23.3	38.6	32.3	32.8	24.5
Reduction of micropores Volume % 300Å ~ 15μ	21.1	16.8	14.4	13.4	9.6	6.7
Reduction of micropores Volume % ~ 300Å	78.9	83.2	85.6	86.6	90.4	93.3
Reduction of micropores Volume % 60 ~ 300Å	3.8	—	6.9	8.9	1.6	10.4
Reduction of micropores volume % 30 ~ 60Å	6.7	7.3	6.3	7.6	6.8	7.3
Reduction of micropores volume % 12 ~ 30Å	33.0	34.0	36.2	37.5	30.0	29.0
Reduction of micropores volume % ~ 12Å	35.4	45.0	36.2	32.6	52.0	46.6

Table 6.12 Determination of Exhausted Gas from the Furnace

Position Items		Outlet of furnace	Outlet of after heating room	Outlet scrubber	Method used for analysis
Temperature of gas	°C	240	620	40	TC-Thermometer
Dry gas flow	Nm^3/H	69	675	373	JIS Z-8808
Moisture	Vol %	42.4	22.5	2.4	JIS Z-8808
Soot	g/Nm^3	1.31	0.16	0.05	JIS Z-8808
SO_2	ppm	3	235 ~ 240	6 ~ 9	JIS K-0103
NO	ppm	62 ~ 74	100 ~ 110	113 ~ 123	JIS K-0104
NOX	ppm	65 ~ 76	105 ~ 115	118 ~ 128	
Odor PO	–	11.9	2.6	5.3	Equilibrium method with salt water
CO_2	Vol %	10.9	11.4	8.3	Orsat Anatyzing method
O_2	Vol %	3.3	4.0	8.6	


CT
2CT
3CT
4CT
5CT
6CT
V-4
2V-4
3V-4
4V-4
5V-4
6V-4
Lsw-1
2Lsw-1
3Lsw-1
4Lsw-1
5Lsw-1
6Lsw-1
40A
50A
V-1-2
V-1-3
2V-1-2
3V-1-2
3V-1-3
4V-1-3
4V-1-2
5V-1-2
5V-1-3
6V-1-2
V-1-1
V-5
V-6
2V-5
2V-6
3V-5
3V-6
4V-5
4V-6
5V-5
5V-6
6V-5
6V-6
V-2
V-3
2V-2
2V-3
3V-2
3V-3
4V-2
4V-3
5V-2
5V-3
6V-2
6V-3
100A
10A
30A
MT-2
To A
To B
SP-2
2SP-2
3SP-2
4SP-2
5SP-2
6SP-2
MT-1
2MT-1
3MT-1
4MT-1
5MT-1
6MT-1
Lsw-2
2Lsw-2
3Lsw-2
4Lsw-2
5Lsw-2
6Lsw-2
Lsw-3
Lsw-4
Lsw-5
S
2S
3S
4S
5S
6S
SP-1
P-5

Mark	Equipments	Mark	Equipments	Mark	Equipments	Mark	Equipments
CT	Carbon Contactor	P-4	Pump for SW	V-6	Valves for BWW	Lsw-4	WLS for PW Holding Tank
S	Intermediate PW Holding Tank	SP-1	Sampling Pumps for FW	P-5	Pump for BWW Drain	Fi-1	FI for BW
MT-1	FMT for PW	SP-2	Sampling Pumps for PW	P-6	Handing Water Pump	Fi-2	FI for SW
MT-2	FMT for BWW	V-1	Valves for FW	Sol-1	Solenoid Samplers	Fu-1	FS for BW
HF	FW Sample Hold ing Tank	V-2	Valves for PW	Sol-2	Solenoid Samplers	Fu-2	FS for SW
P-1	Feed Pump	V-3	Valves for BW	Lsw-1	WLS for CT	Fi-3	FI for FW
P-2	Intermediate Feed Pump	V-4	Valves for SW	Lsw-2	WLS for S	Fu-3	FS for FW
P-3	Pump for BW	V-5	Valves for Headloss Rage, WS	Lsw-3	WLS for FW Holding Tank	Lsw-5	WLS for Drain Pit

(Abbreviations) FW: Feed Water, PW: Carbon Product BW: Backwash Water, SW: Surface-wash Water, BWW: Backwash Waste Water, FMT: Flow Measuring Tank, WLS: Water Level Switch, FI: Flow Indicator, FS: Flow Meter Senser

Fig. 6.1 Flow Chart of Carbon Contactor

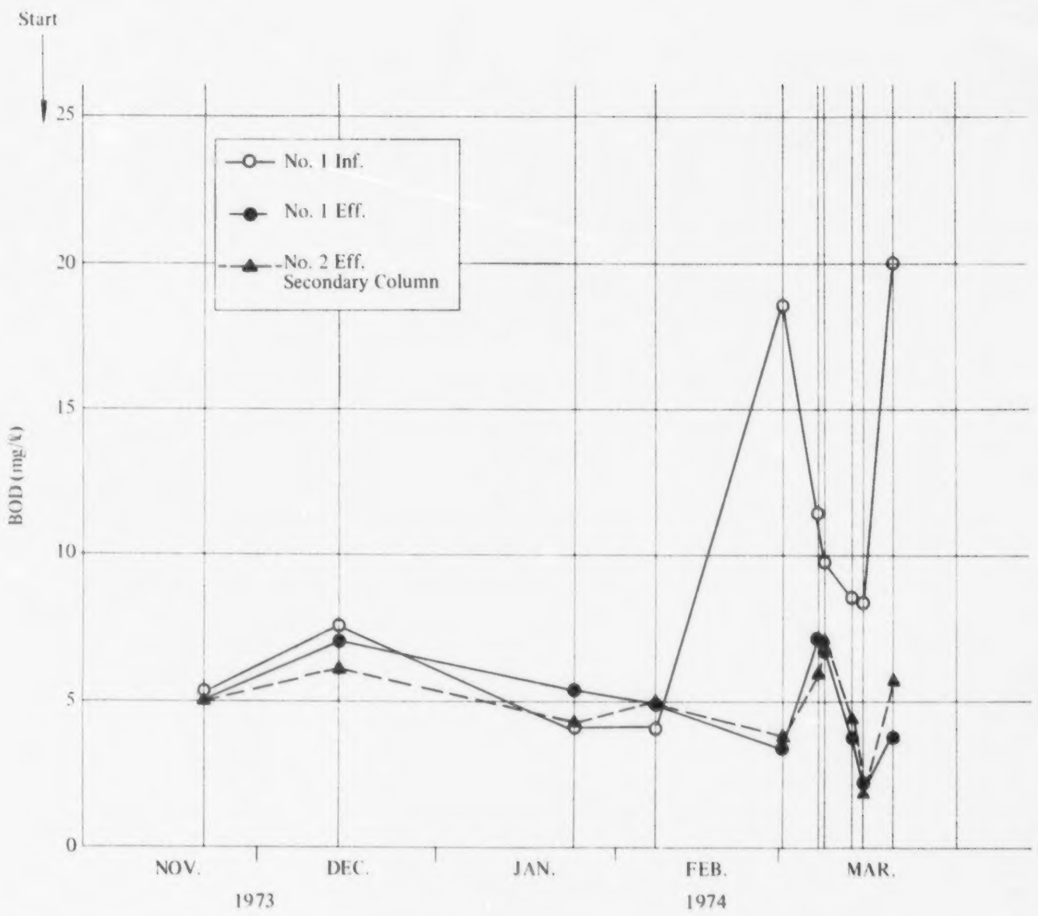


Fig. 6.2 (a) BOD Adsorption by Granular Activated Carbon of Column No. 1 and No. 2

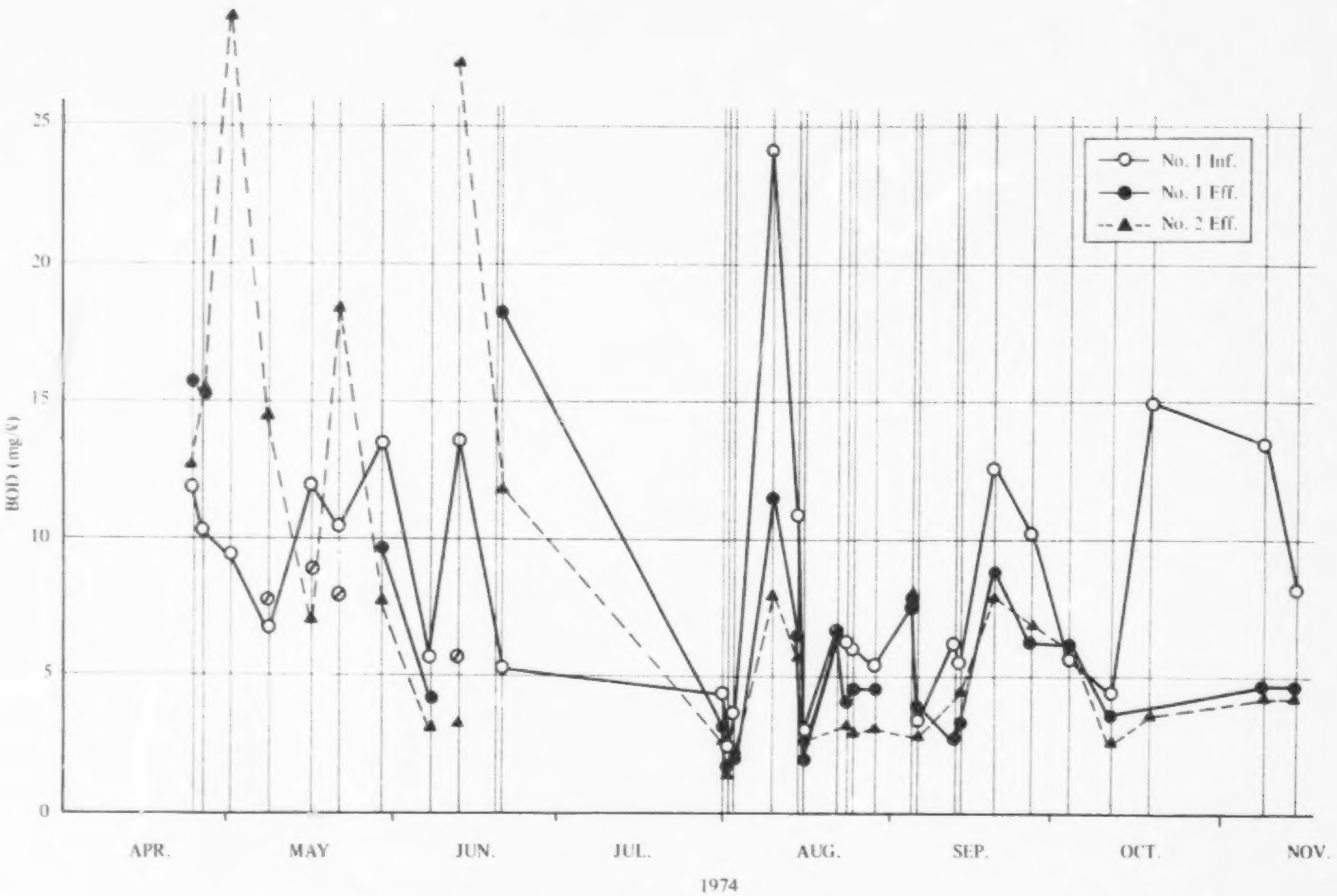


Fig. 6.2 (b) BOD Adsorption by Granular Activated Carbon of Column No. 1 and No. 2

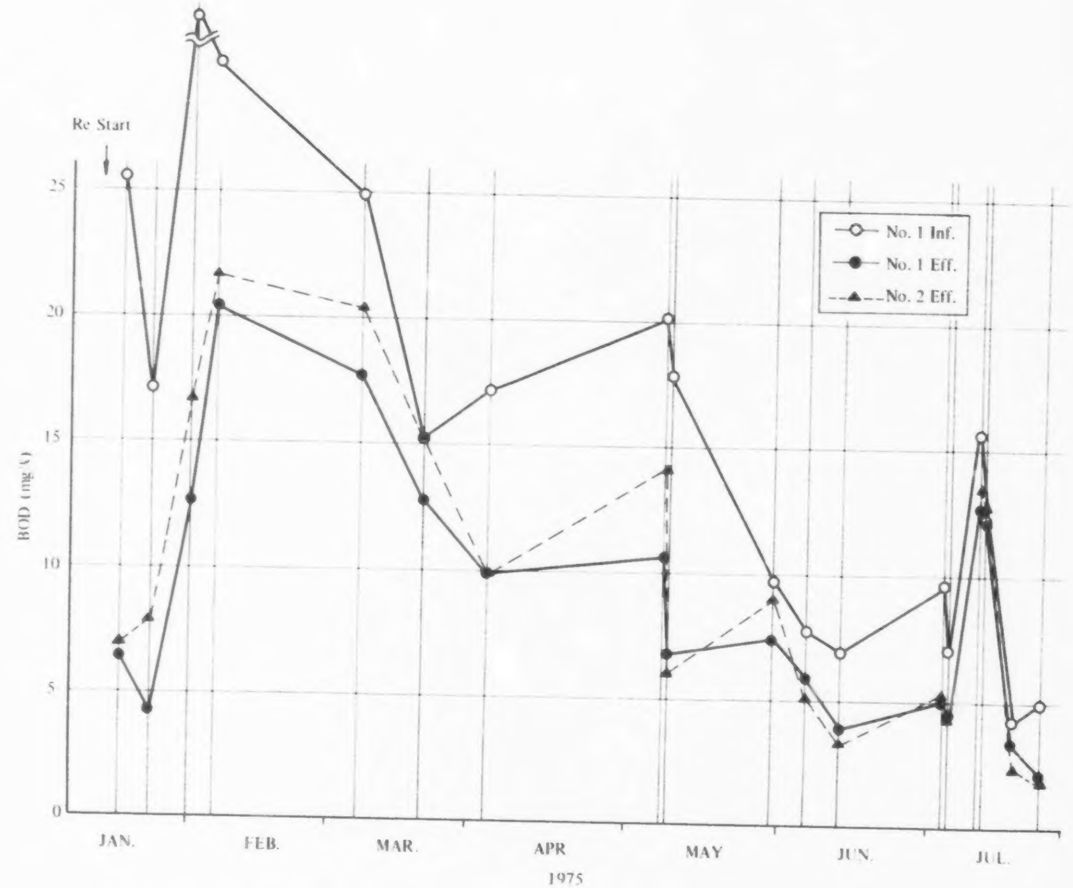


Fig. 6.2 (c) BOD Adsorption by Granular Activated Carbon of Column No. 1 and No. 2

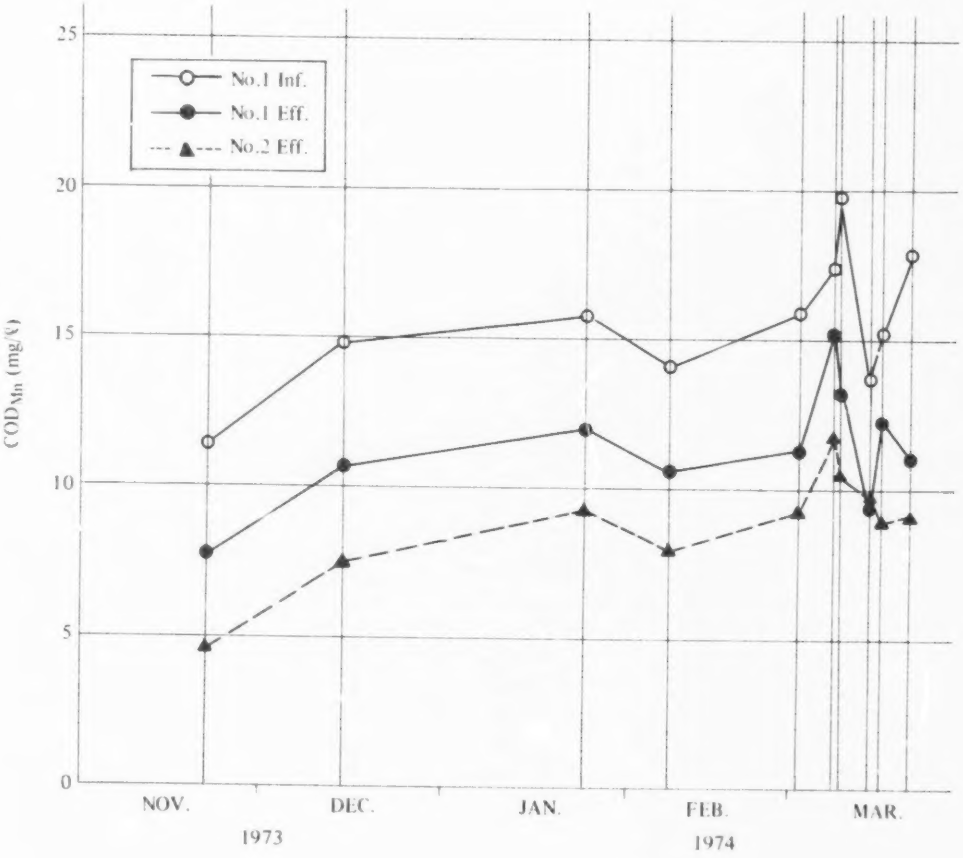


Fig. 6.3 (a) COD$_{Mn}$ Adsorption by Granular Activated Carbon of Column No. 1 and No. 2

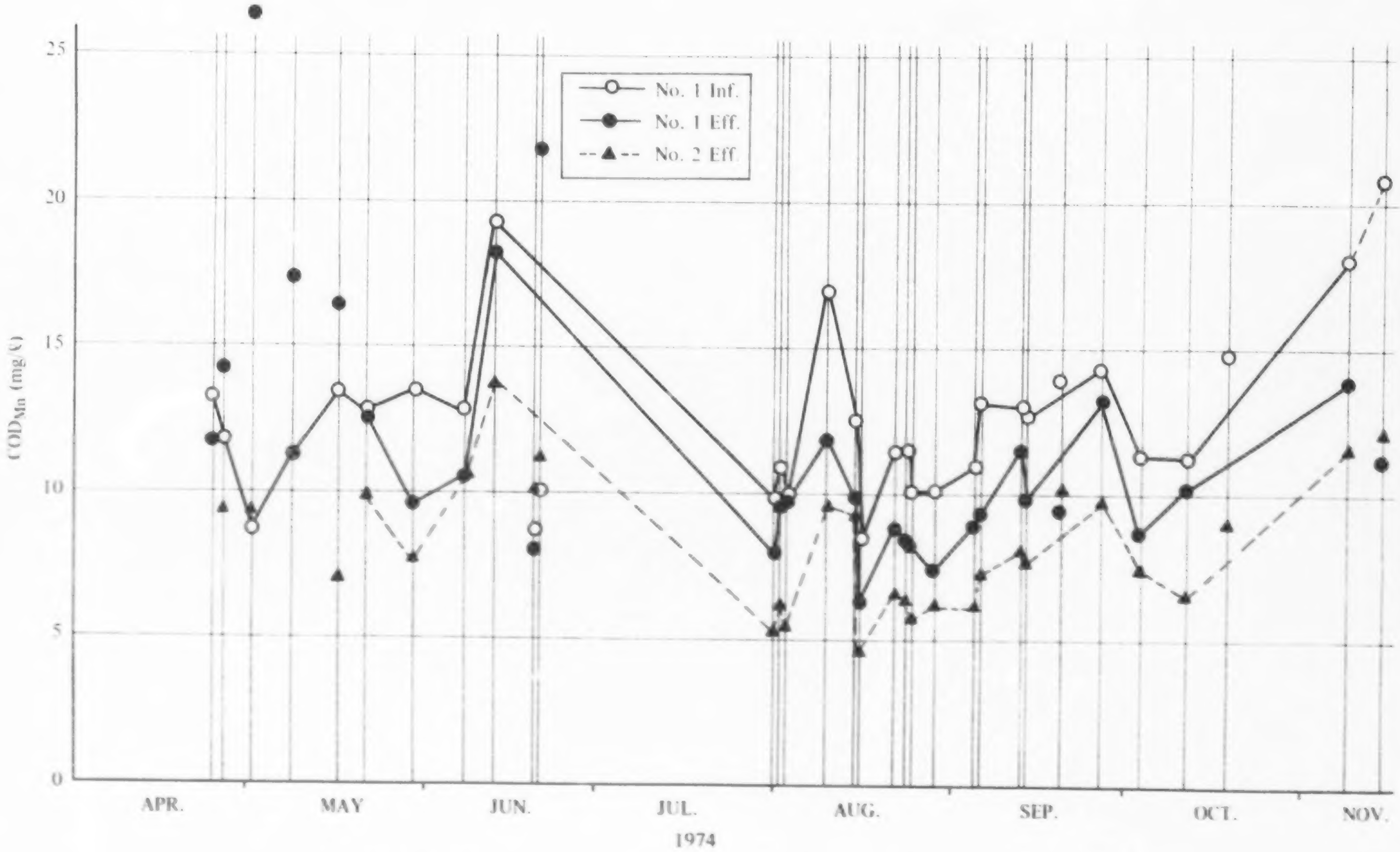


Fig. 6.3 (b) COD_{Mn} Adsorption by Granular Activated Carbon of Column No. 1 and No. 2

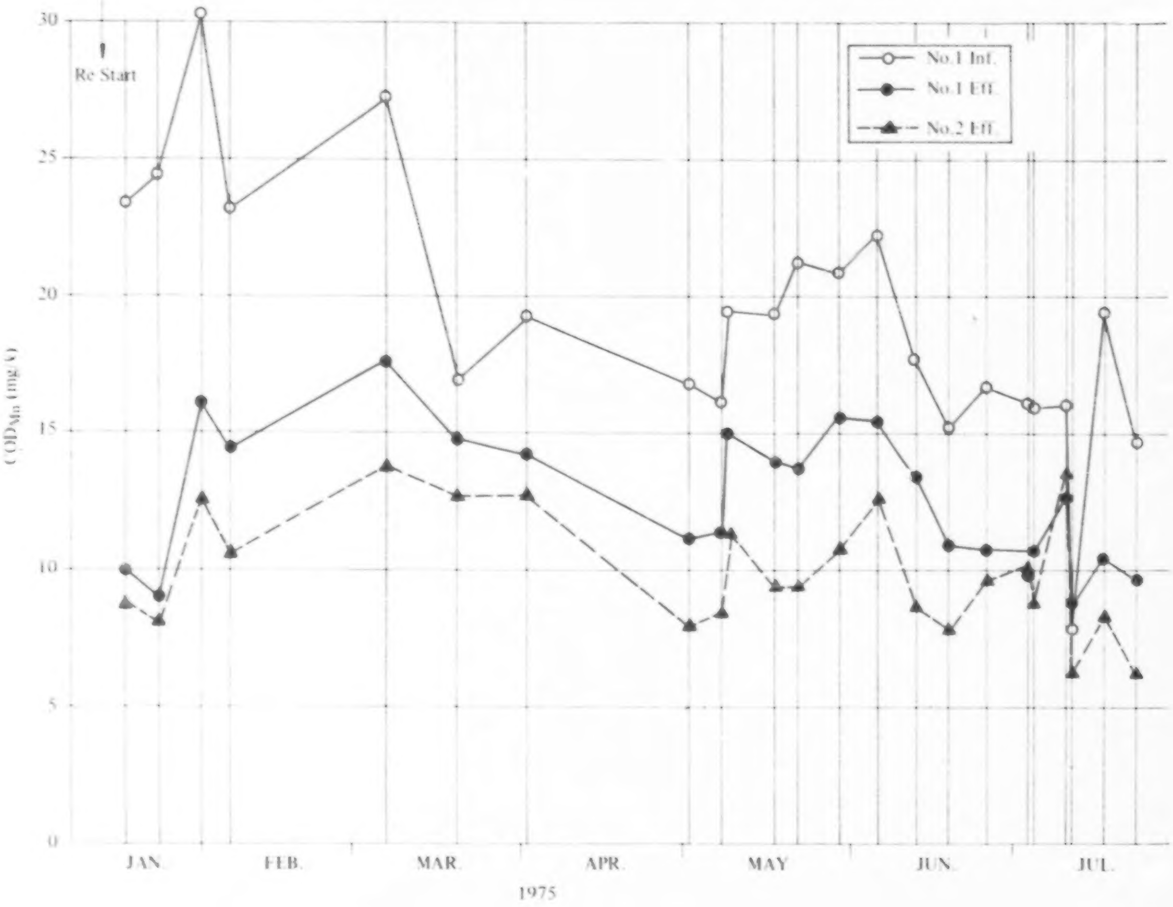


Fig. 6.3 (c) COD$_{Mn}$ Adsorption by Granular Activated Carbon of Column No. 1 and No. 2

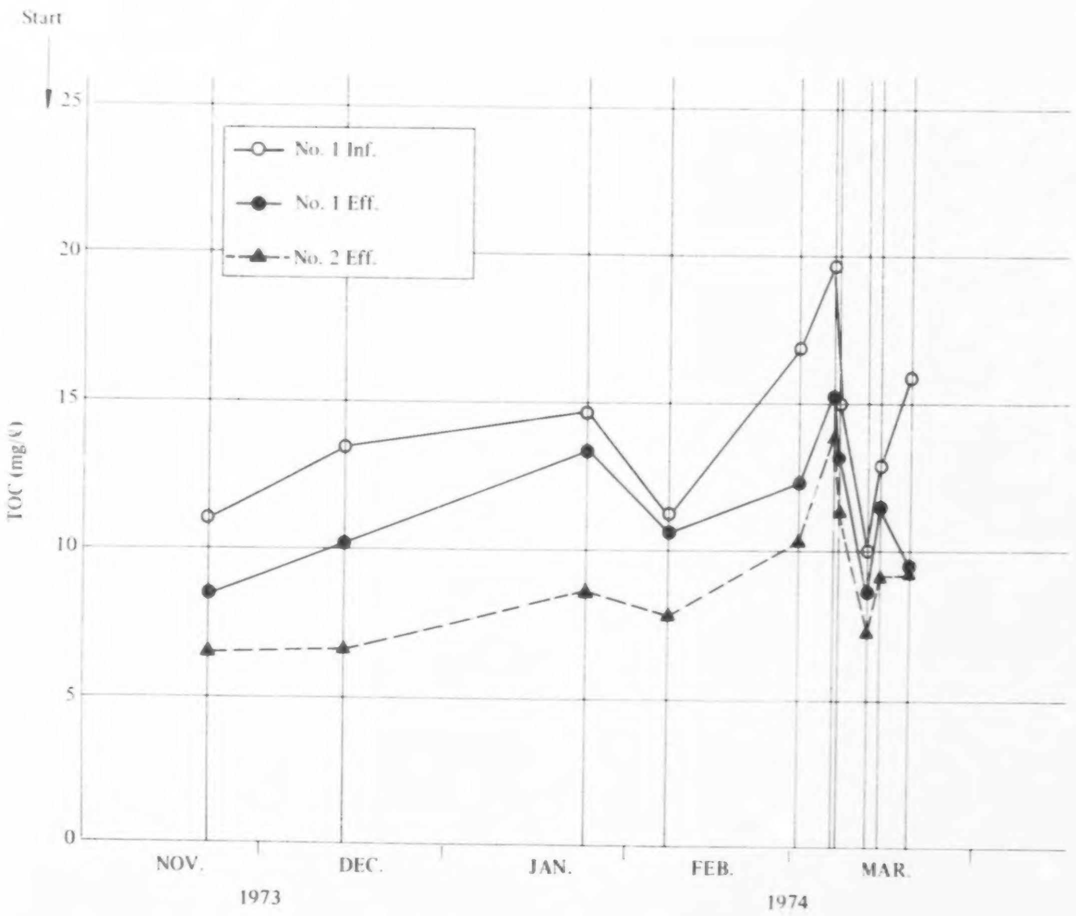


Fig. 6.4 (a) TOC Adsorption by Granular Activated Carbon of Column No. 1 and No. 2

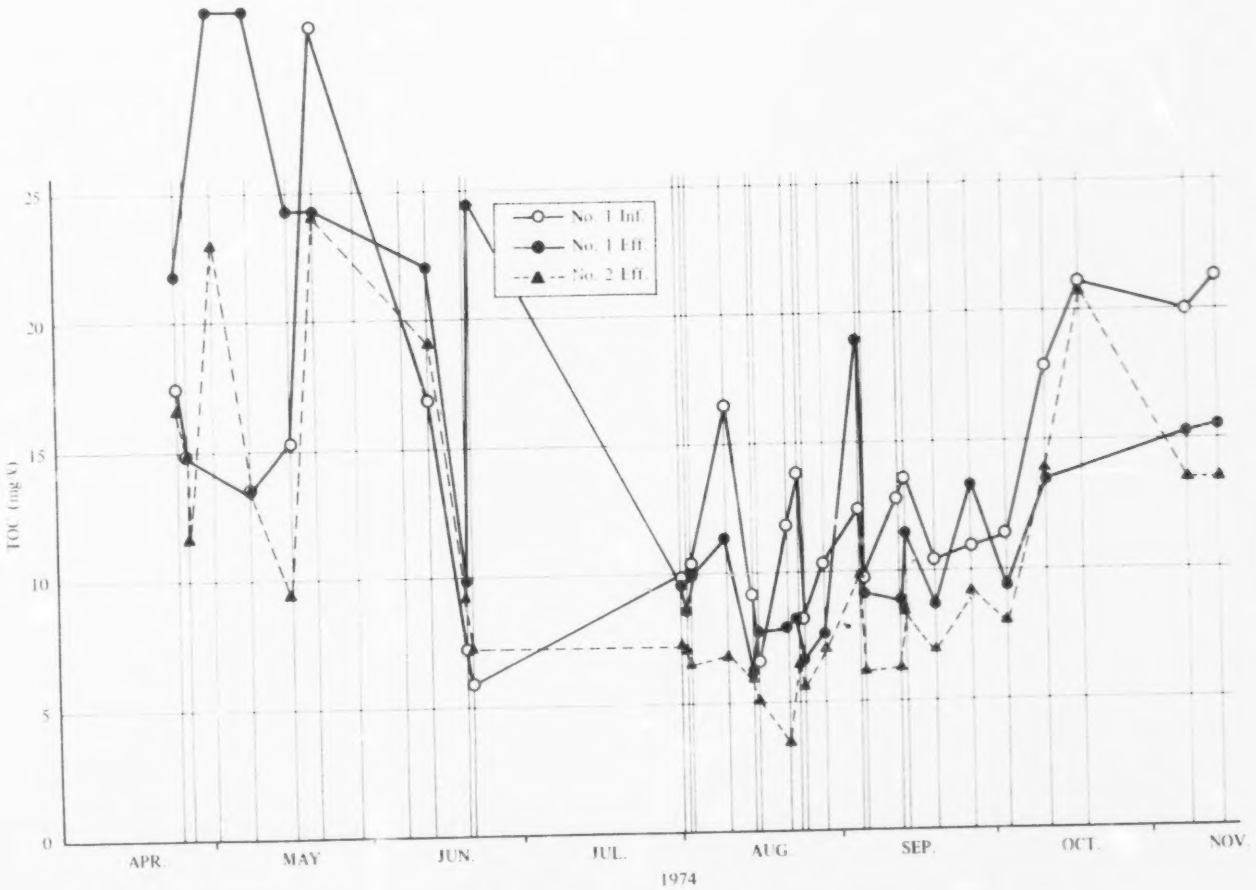


Fig. 6.4 (b) TOC Adsorption by Granular Activated Carbon of Column No. 1 and No. 2

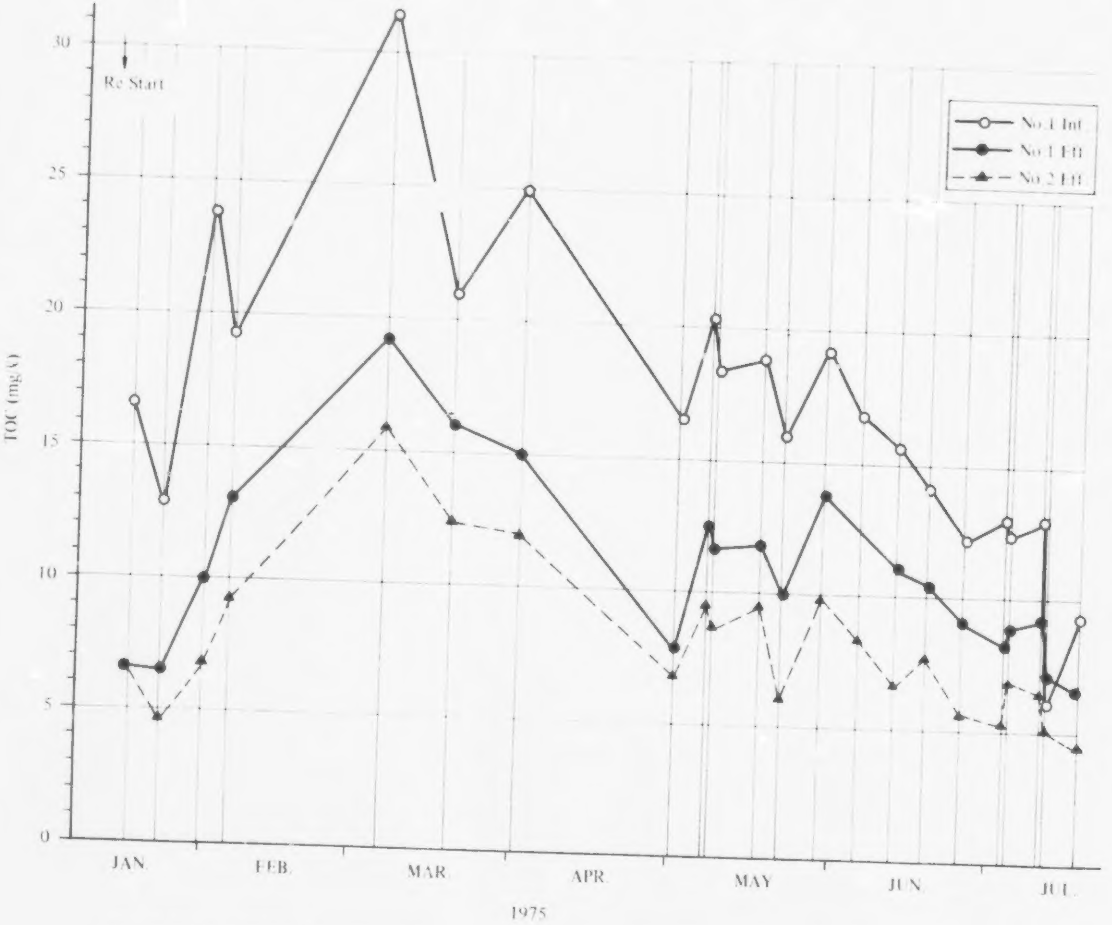


Fig. 6.4 (c) TOC Adsorption by Granular Activated Carbon of Colum No. 1 and No. 2

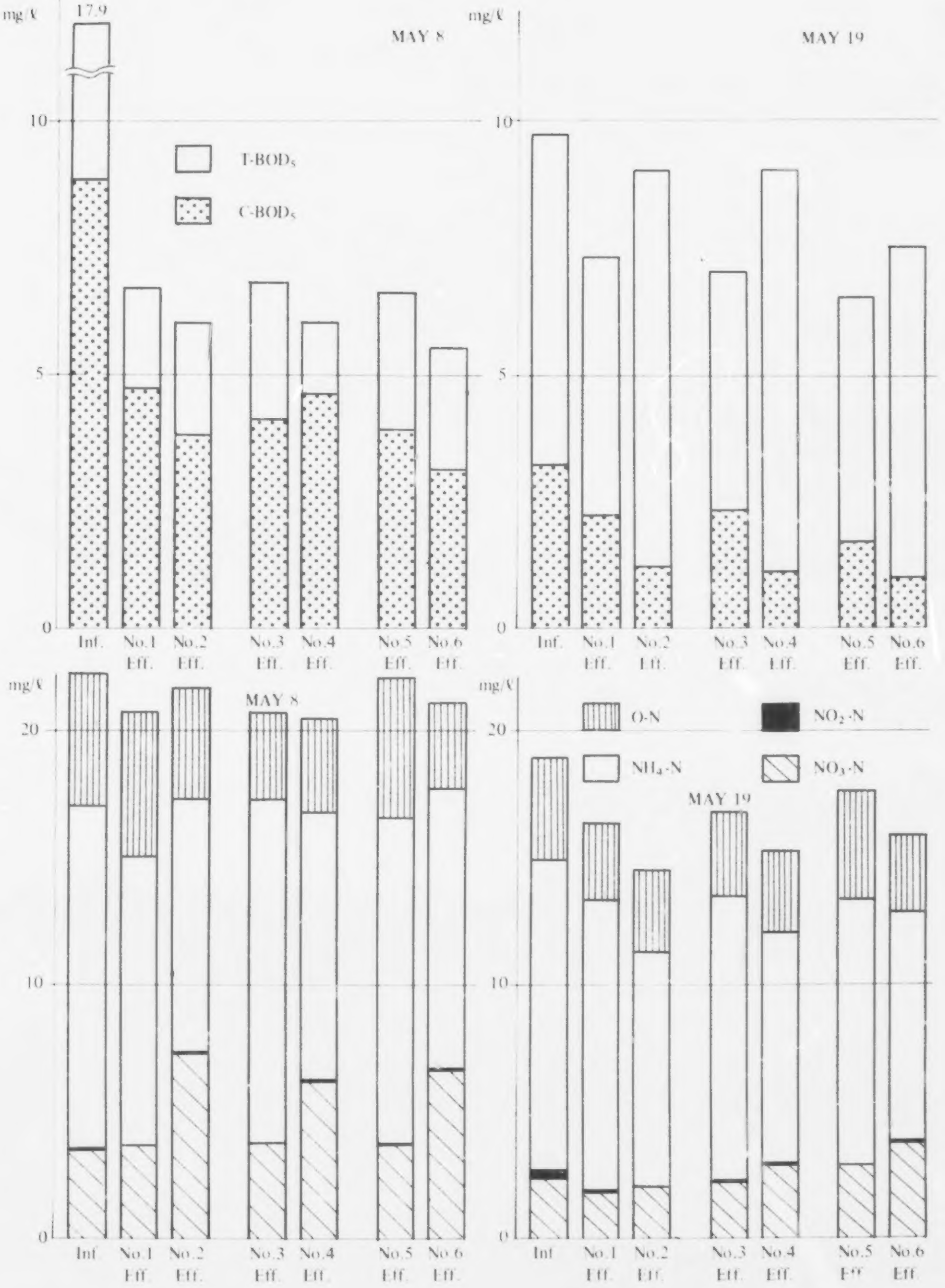


Fig. 6.5 Effect of Nitrification in Contactor on Effluent BOD$_5$

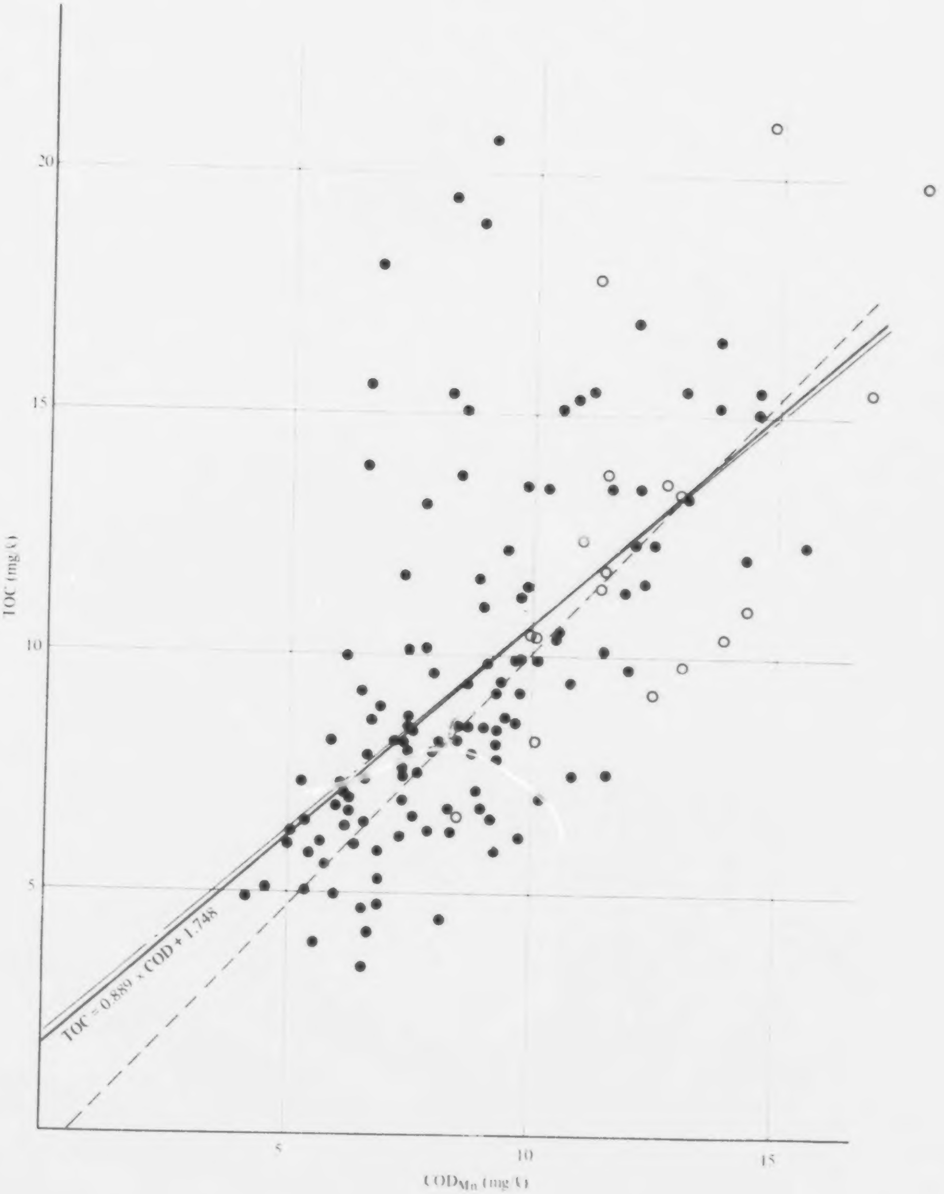


Fig. 6.6 Corelation between COD$_{Mn}$ and TOC

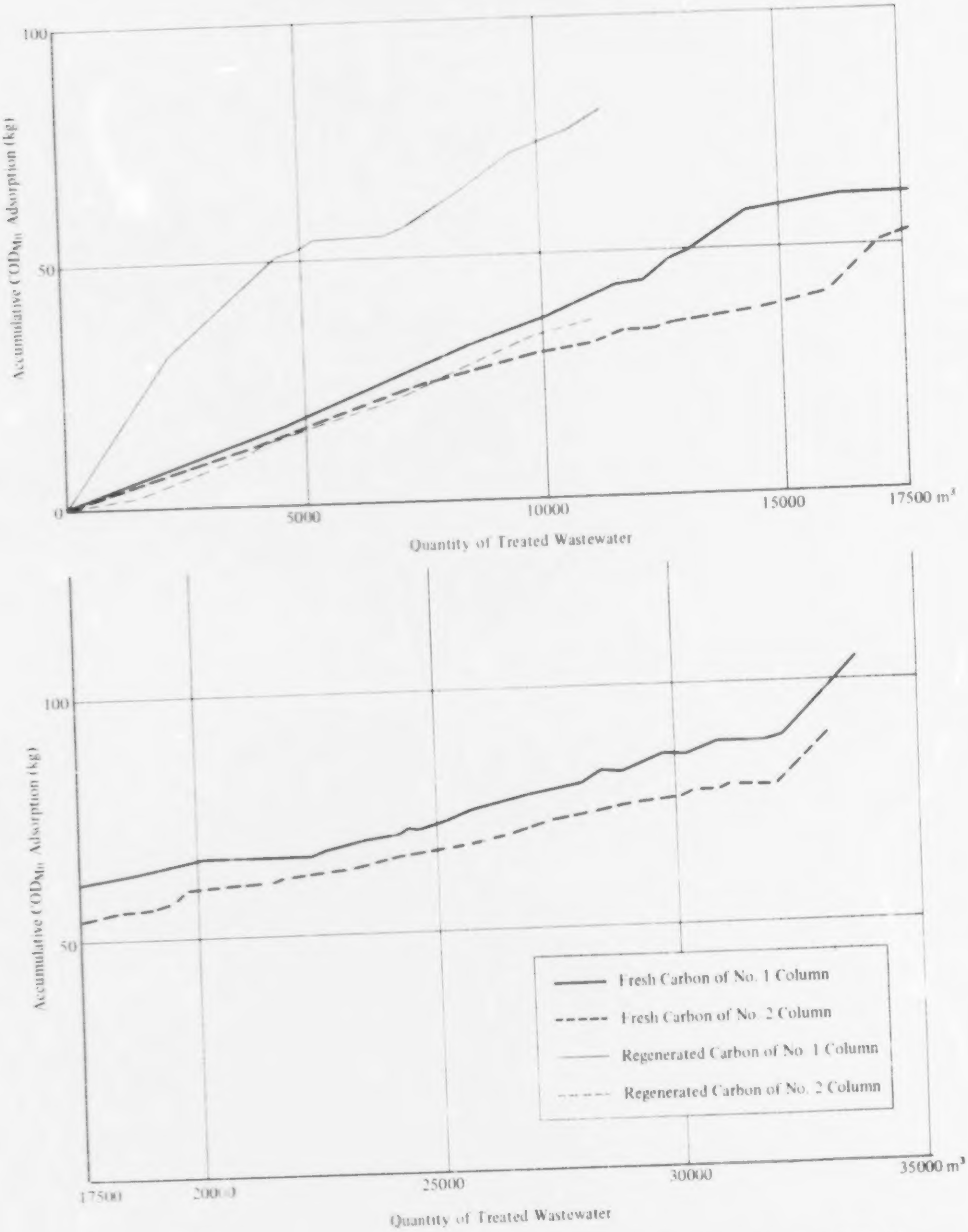
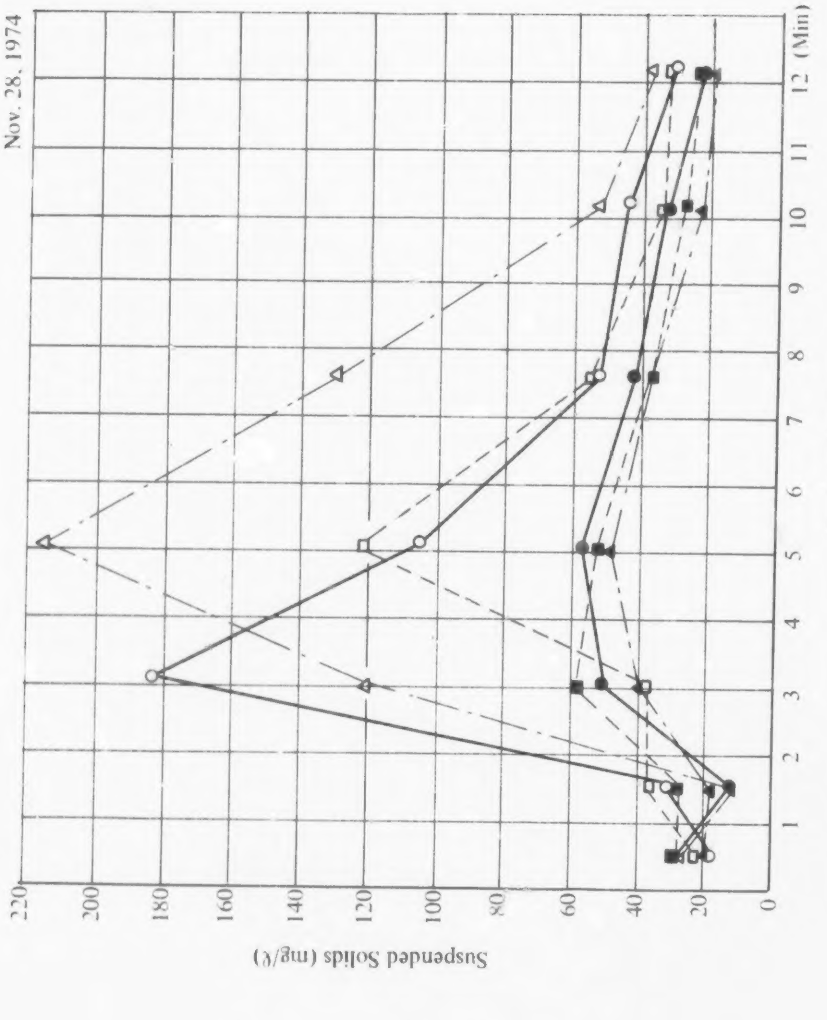


Fig. 6.7 Accumulative COD$_{Mn}$ Adsorption Curve of Column No.1 and No.2



Nov. 28, 1974

Fig. 6.8 SS Variations of Backwash Wastewater

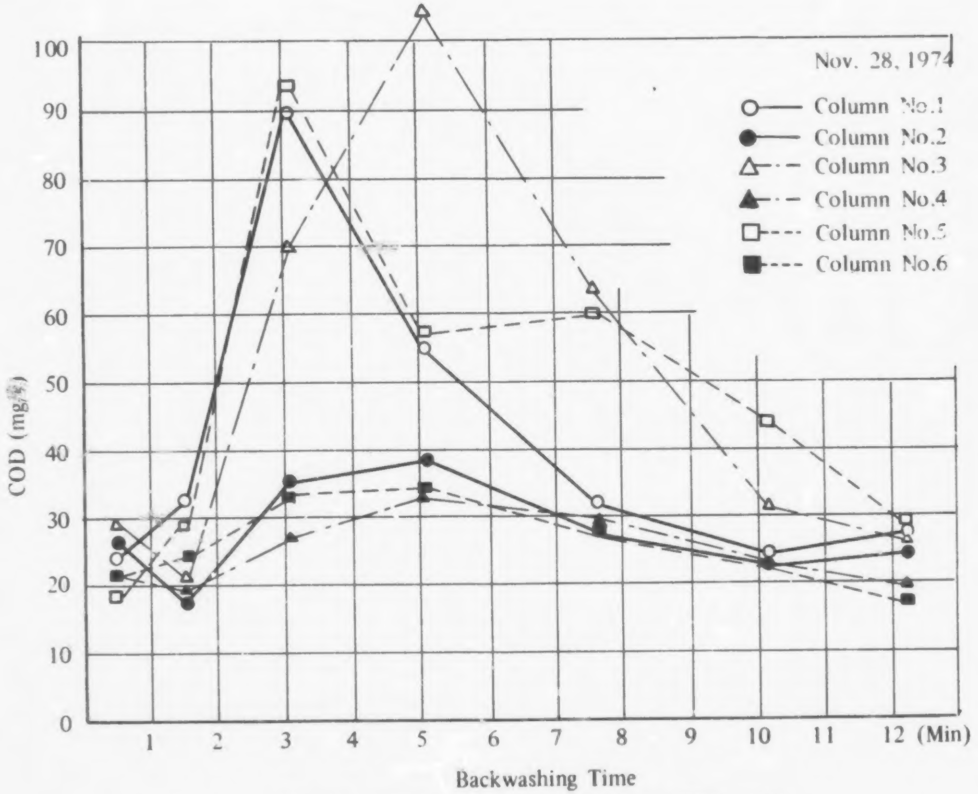


Fig. 6.9 $\mathrm{COD_{Mn}}$ Variations of Backwash Wastewater

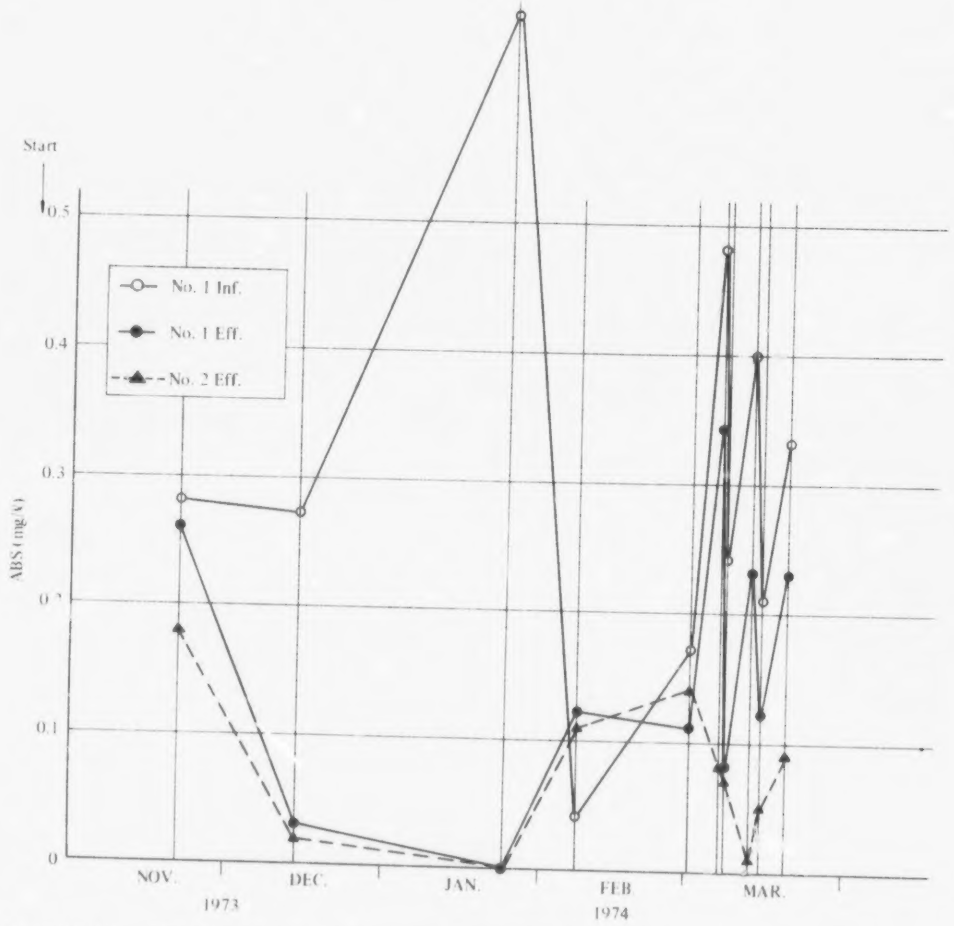


Fig. 6.10 (a) ABS Adsorption by Granular Activated Carbon of Column No. 1 and No. 2

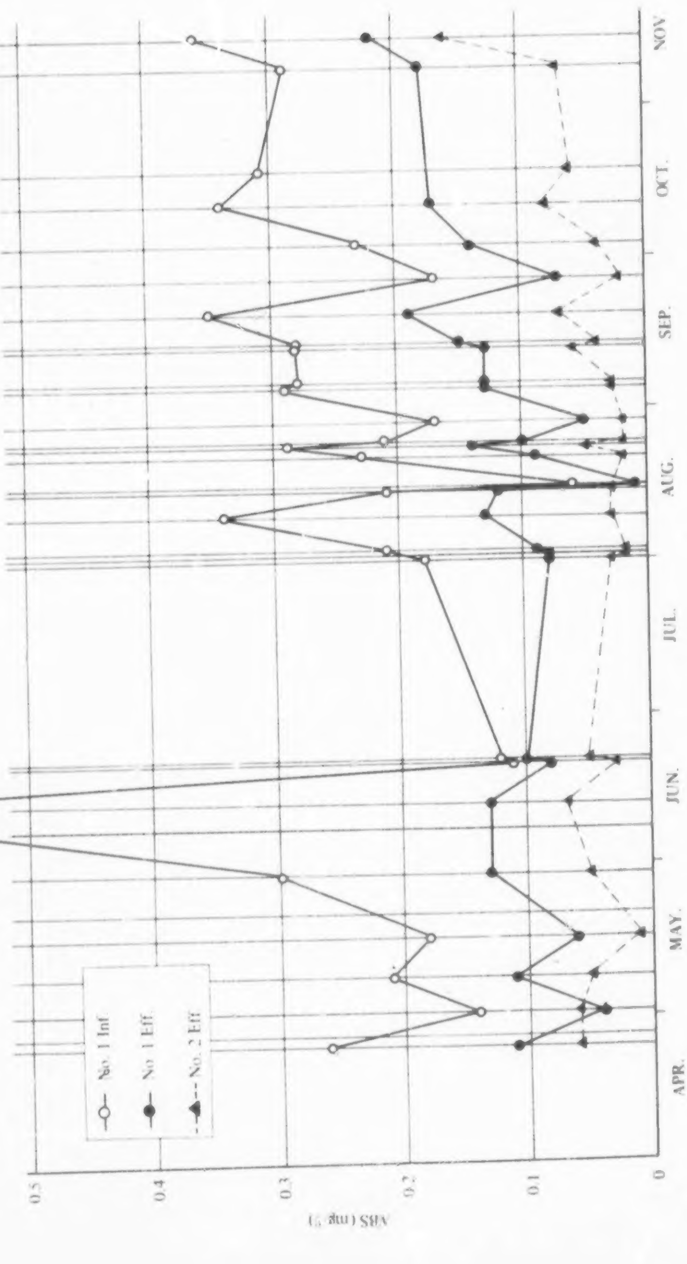


Fig. 6.10 (b) ABS Adsorption by Granular Activated Carbon of Column No. 1 and No. 2

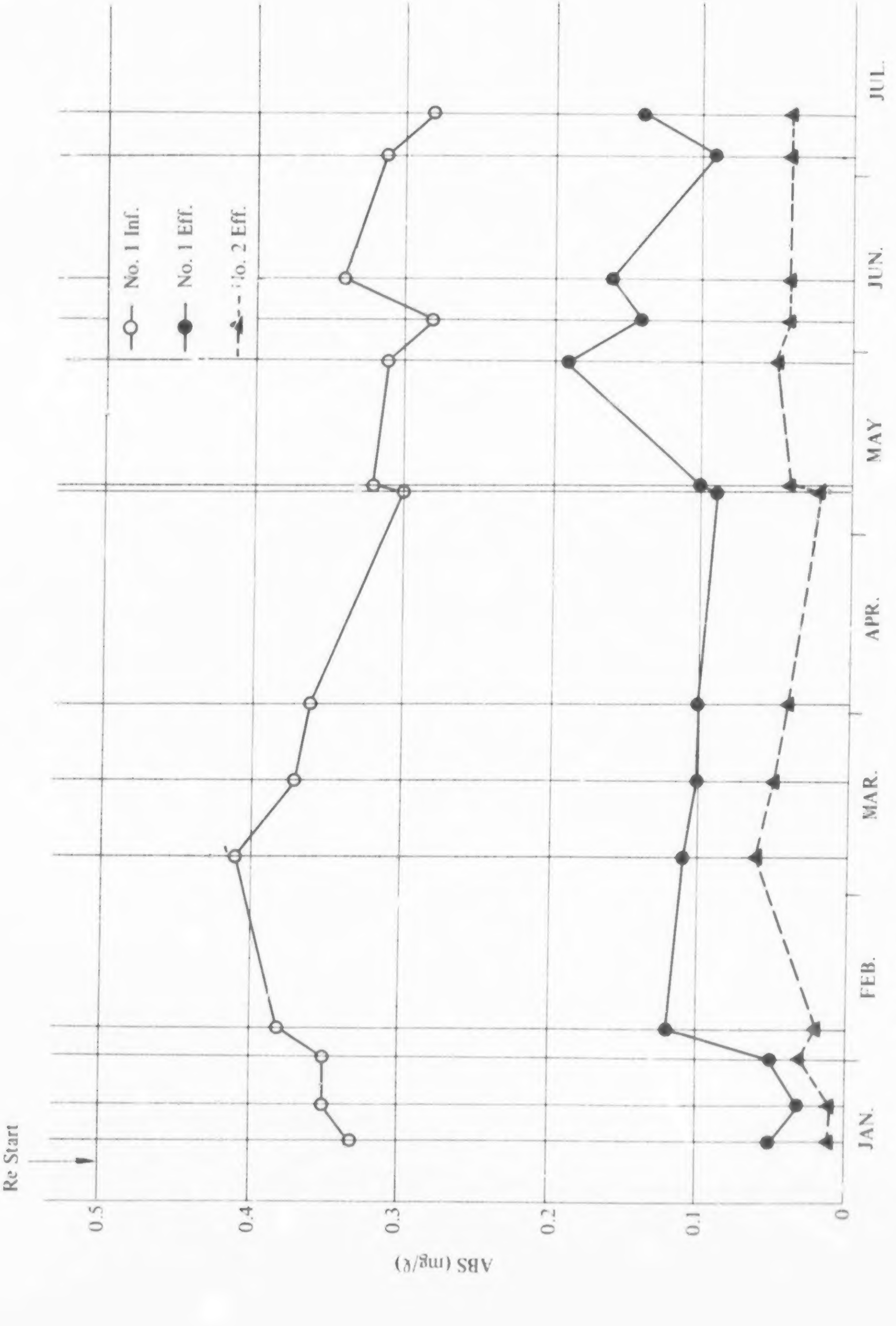


Fig. 6.10 (c) ABS Adsorption by Granular Activated Carbon of Column No. 1 and No. 2

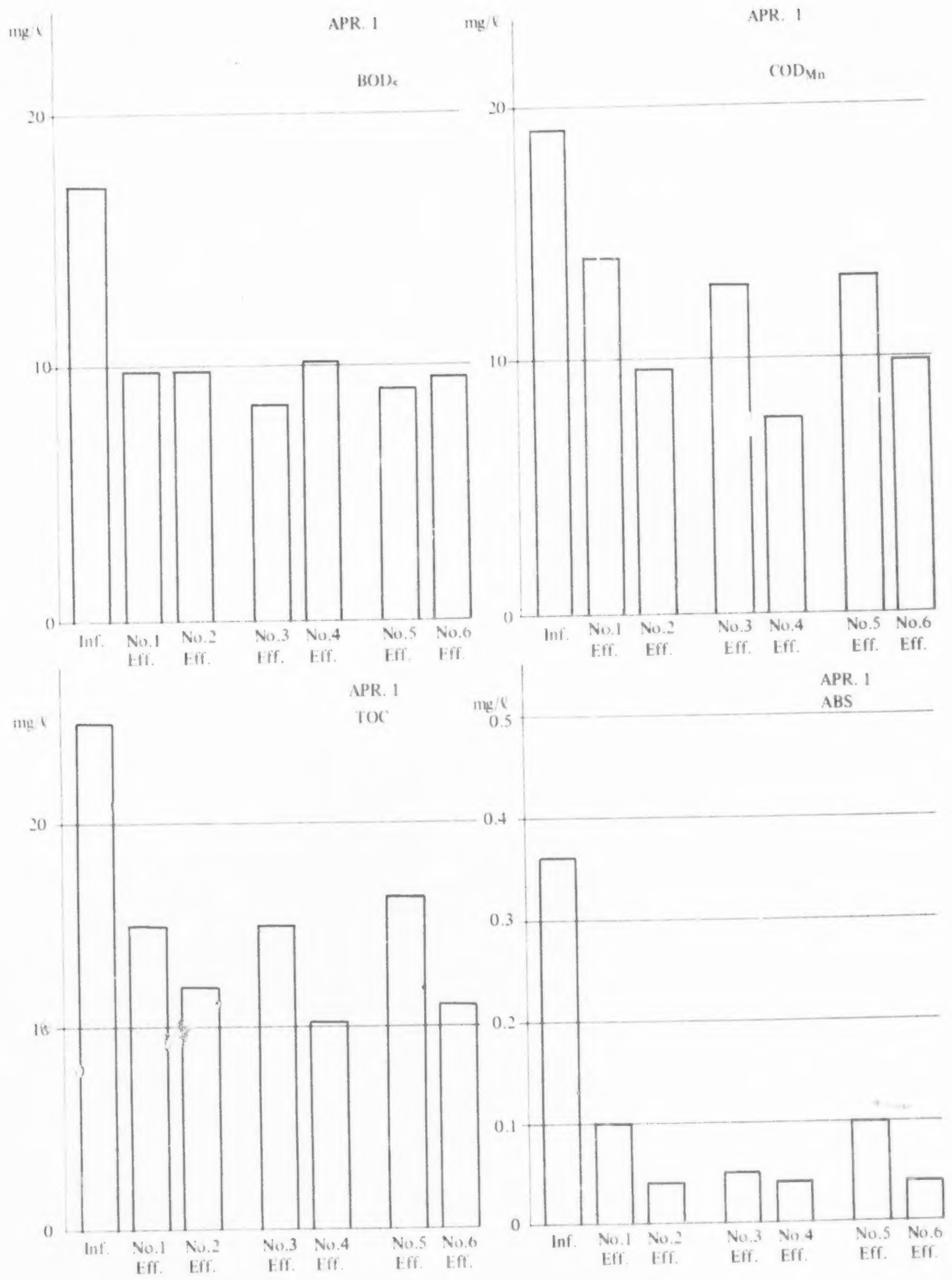
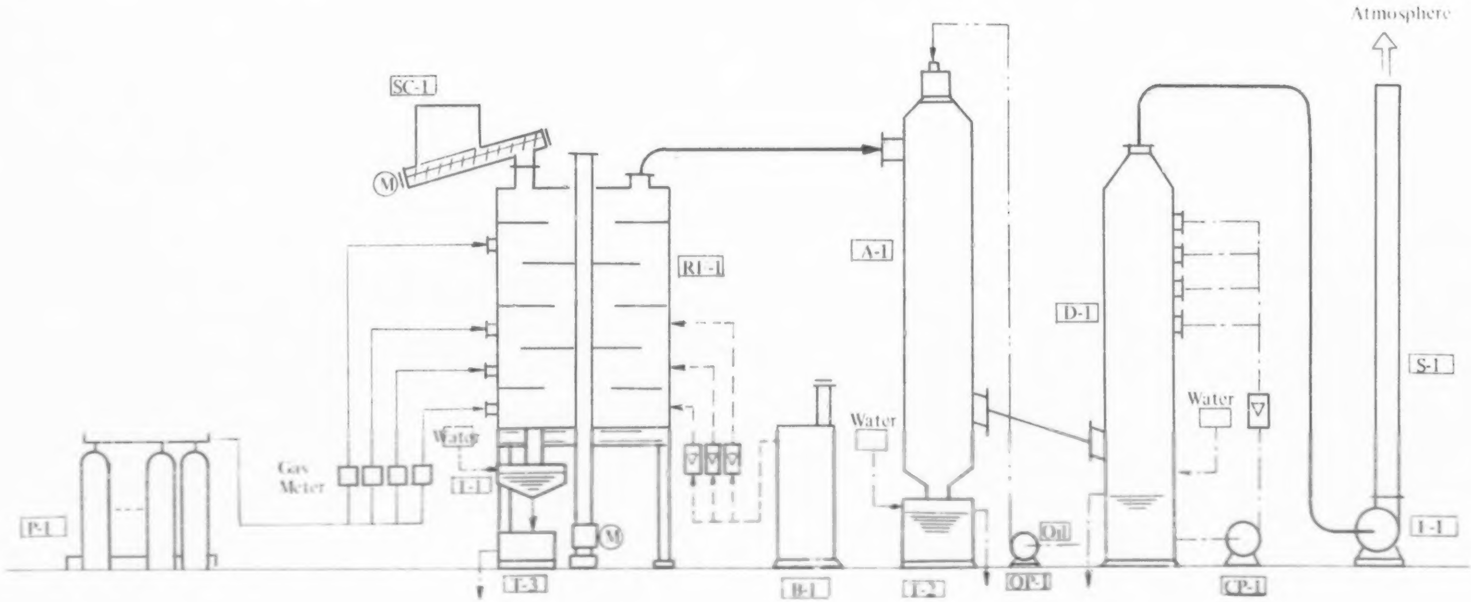


Fig. 6.11 Comparison of Adsorption Effect with Change of Items and Brands


Atmosphere
SC-1
M
RF-1
A-1
D-1
S-1
Water
Water
Water
Oil
Gas Meter
P-1
T-1
T-3
M
B-1
T-2
OP-1
CP-1
F-1

P-1	Fuel Gas	A-1	Exhaust Gas Furnace	T-1	Quench Tank	CP-1	Recycle Pump
SC-1	Screw Conveyer	D-1	Scrubber	T-2	Seal Tank		
RF-1	Regeneration Furnace	S-1	Chimney	T-3	Dewatering Tank		
OP-1	Oil Pump	B-1	Boiler	F-1	Exhasted Gas Fan		

Fig. 6.12 Flow Diagram of the Regeneration System

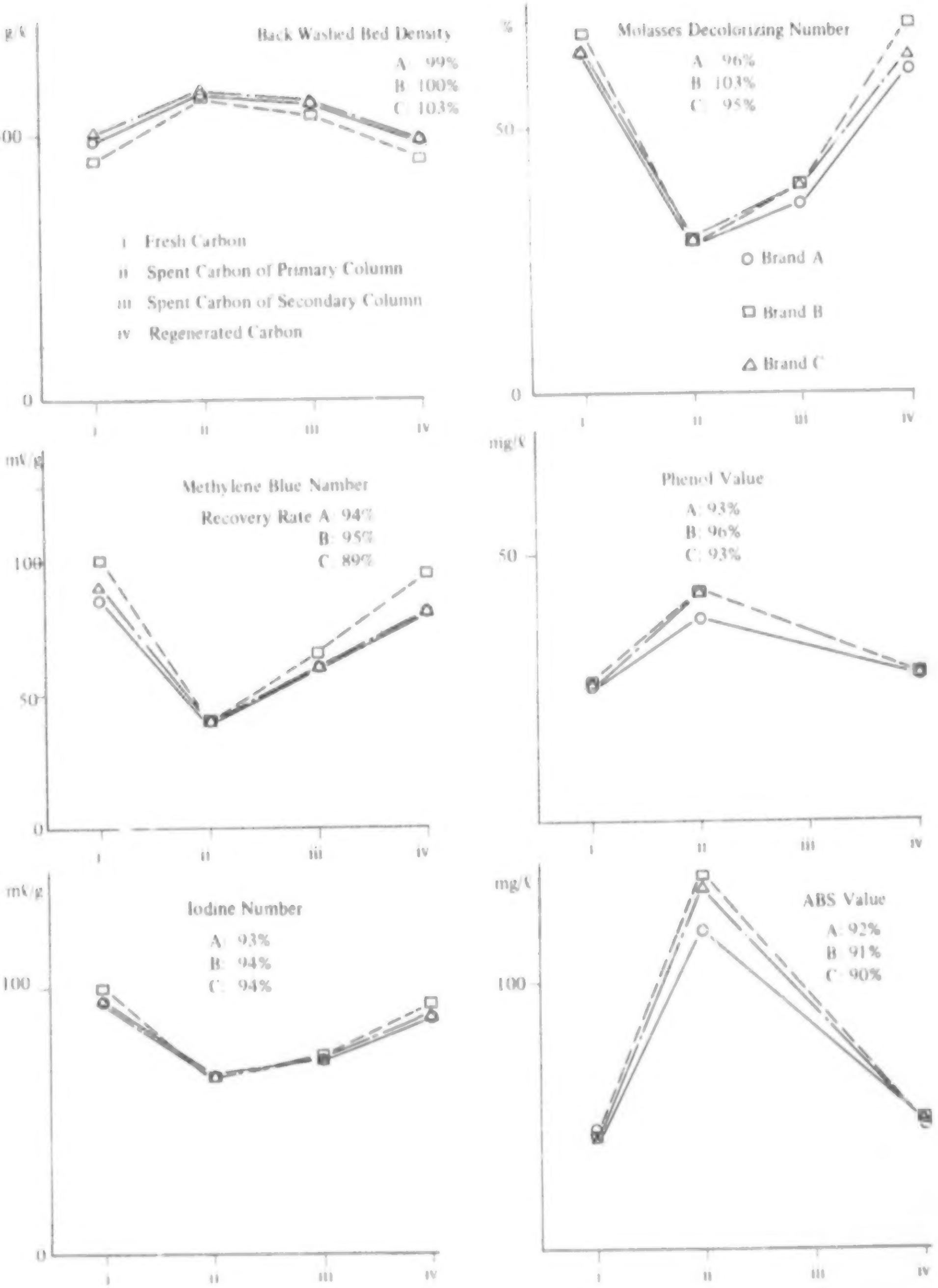


Fig. 6.13 Effect of Regeneration on Some Evaluation Items

Fig. 6.14 TOC Freundrich Line of Brand A Carbon

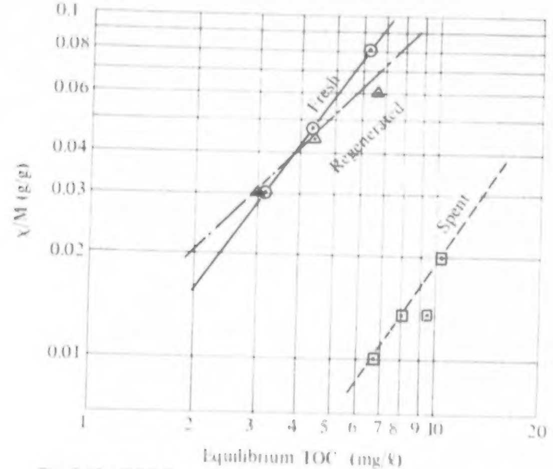


Fig. 6.15 TOC Freundrich Line of Brand B Carbon

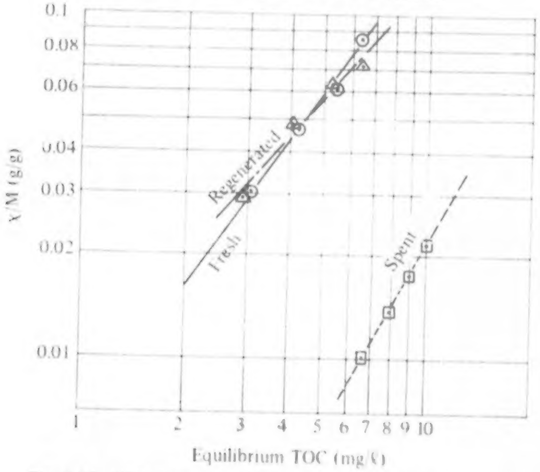
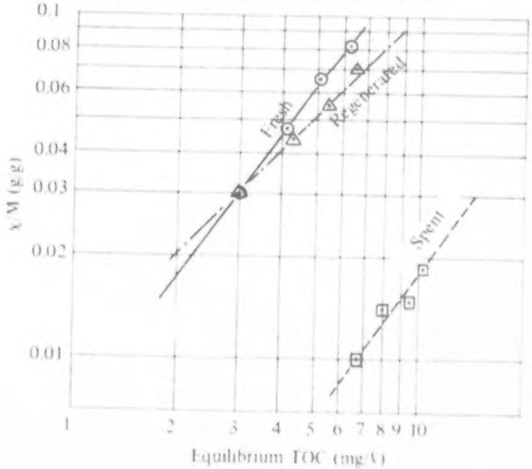


Fig. 6.16 TOC Freundrich Line of Brand C Carbon



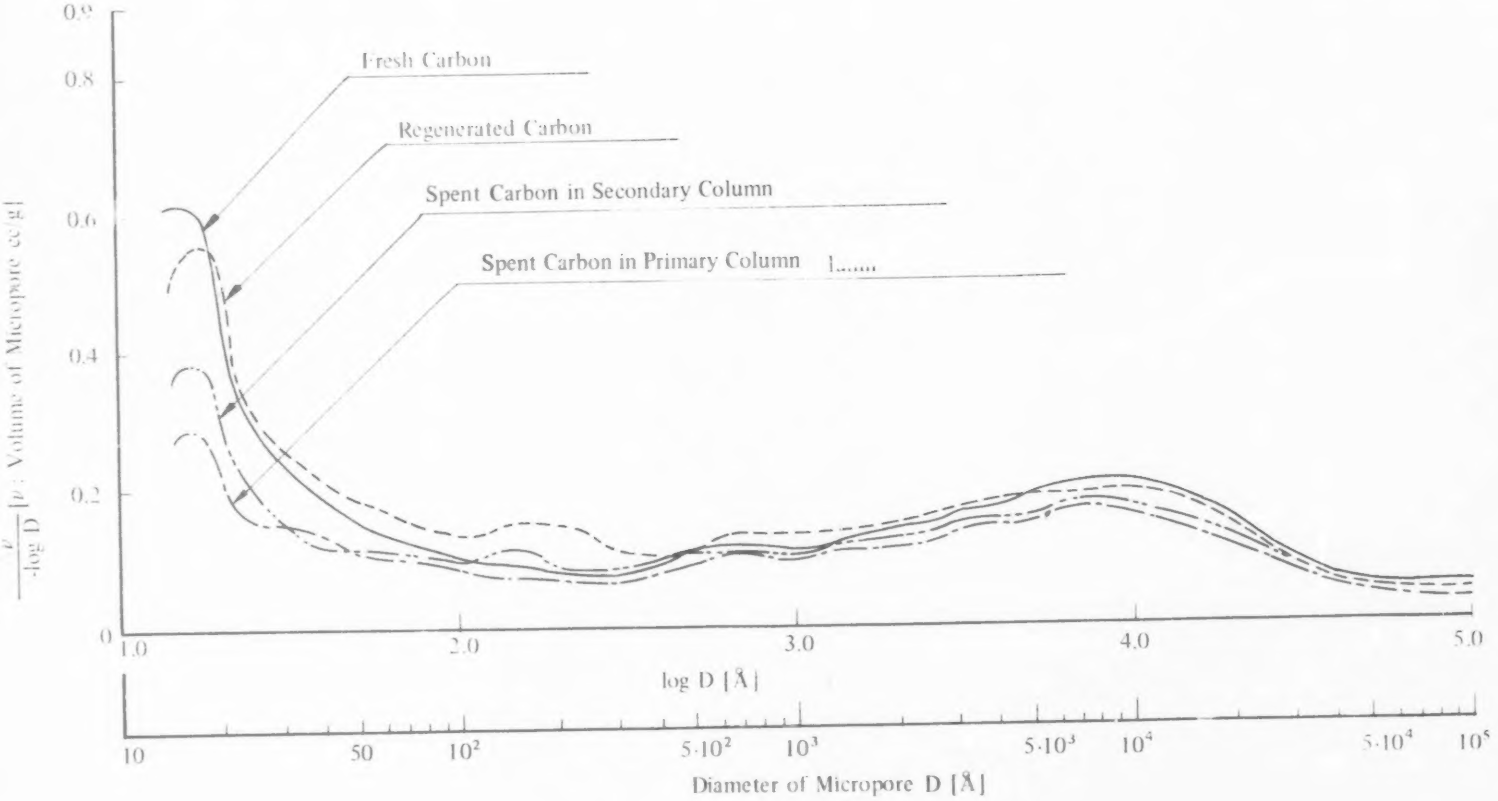


Fig. 6.17 Pore Size Distribution of Brand A Carbon

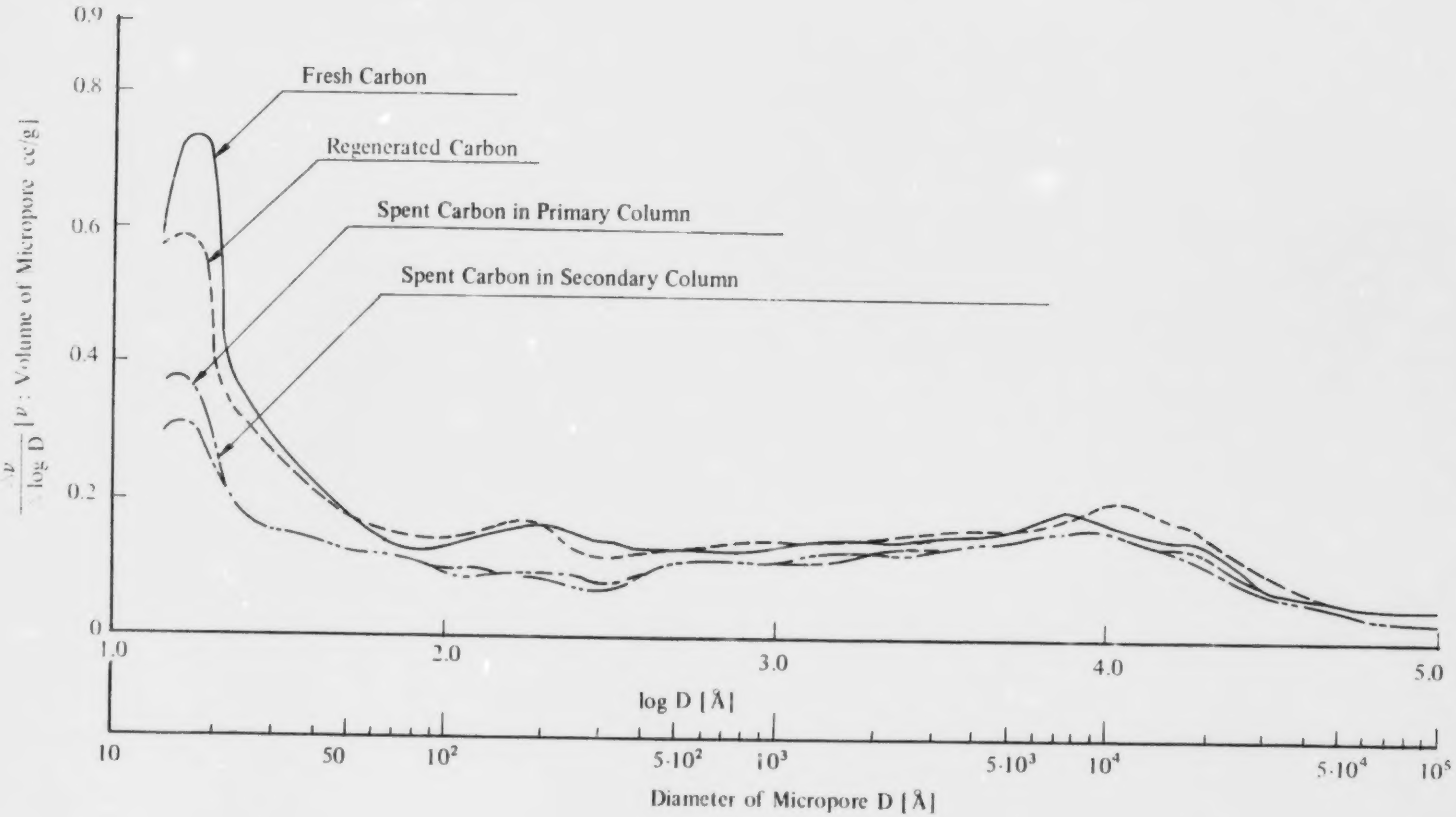


Fig. 6.18 Pore Size Distribution of Brand B Carbon

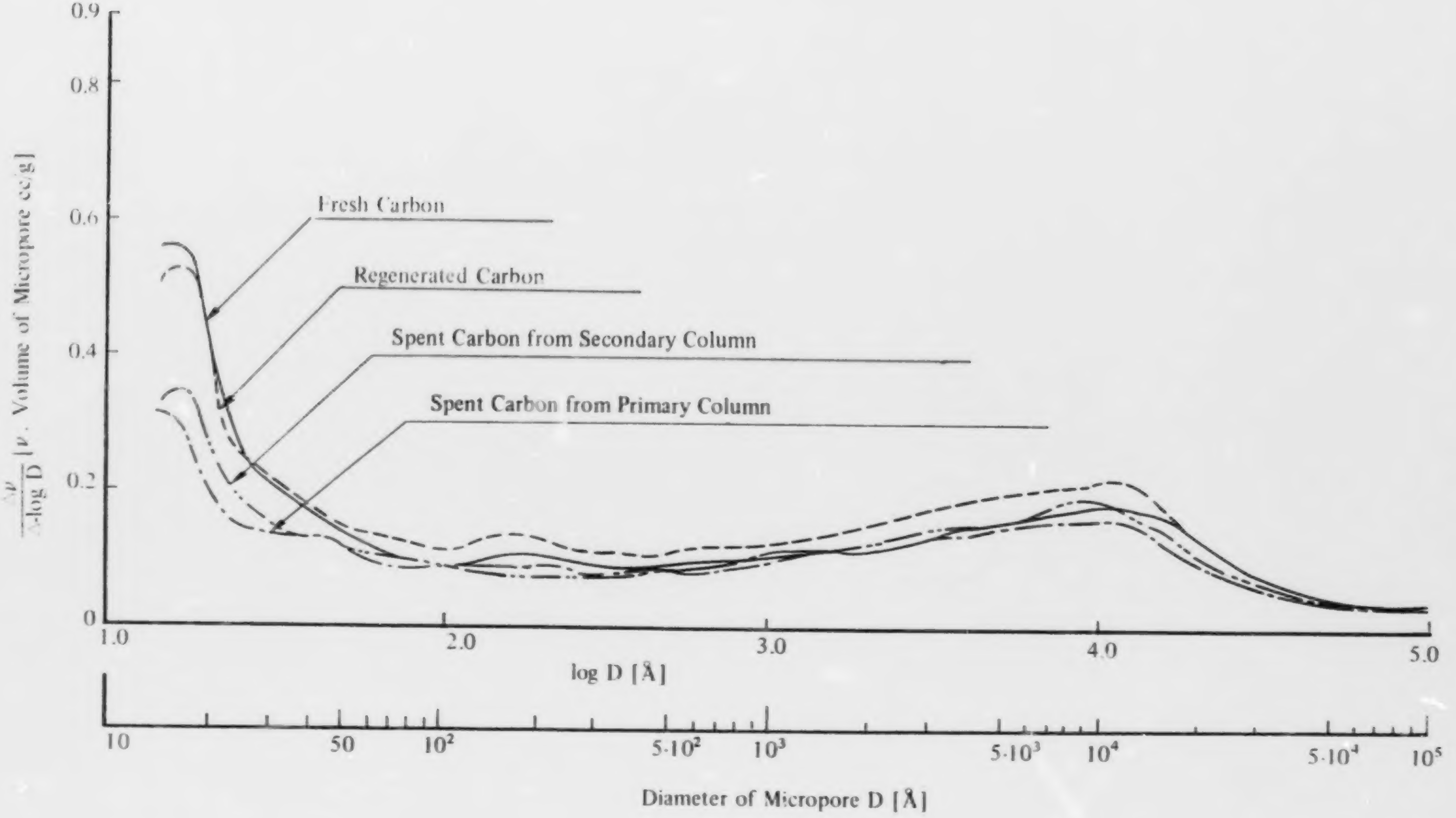


Fig. 6.19 Pore Size Distribution of Brand C Carbon

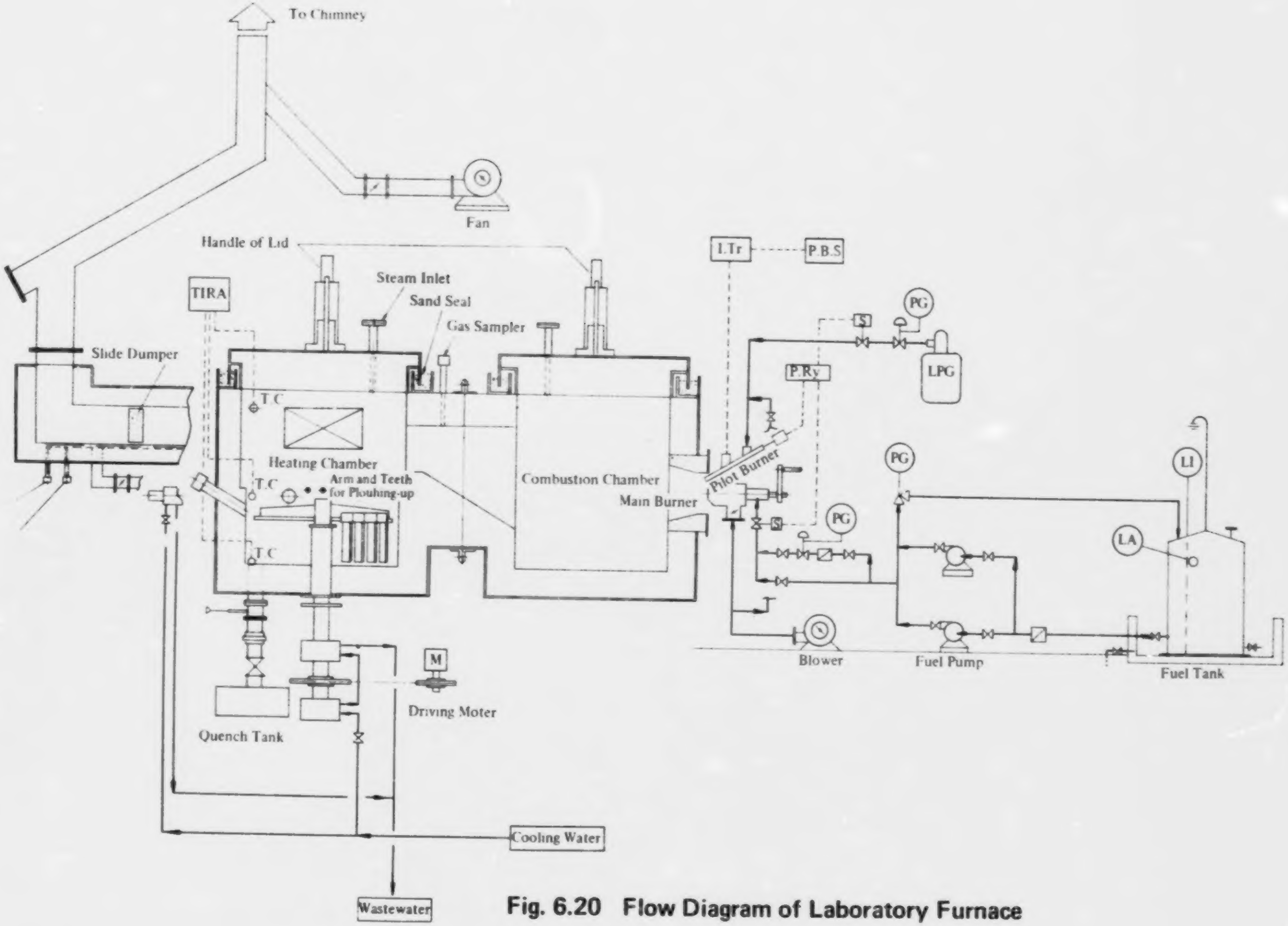

To Chimney
Fan
Handle of Lid
Steam Inlet
Sand Seal
Gas Sampler
TIRA
Slide Dumper
T.C
Heating Chamber
T.C
Arm and Teeth
for Plouhing-up
T.C
Combustion Chamber
Main Burner
Pilot Burner
I.Tr
P.B.S
P.Ry
S
PG
LPG
PG
S
PG
LI
LA
Blower
Fuel Pump
Fuel Tank
M
Driving Moter
Quench Tank
Cooling Water
Wastewater

Fig. 6.20 Flow Diagram of Laboratory Furnace

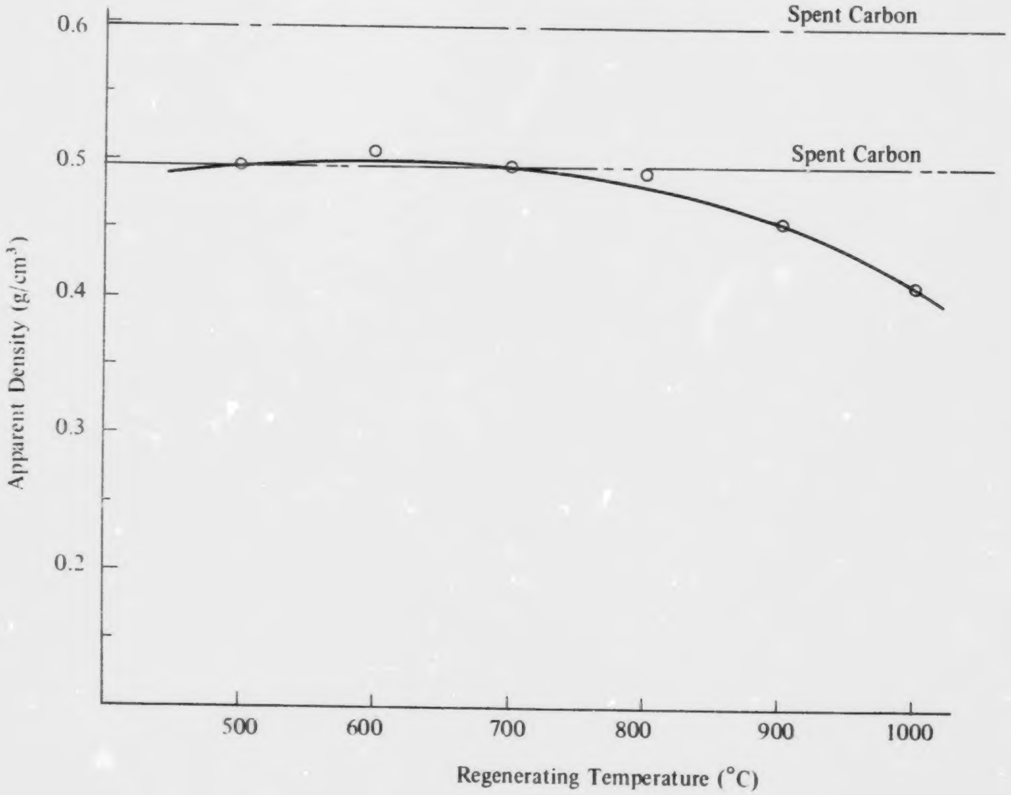


Fig. 6.21 Regenerating Temperature and Apparent Density

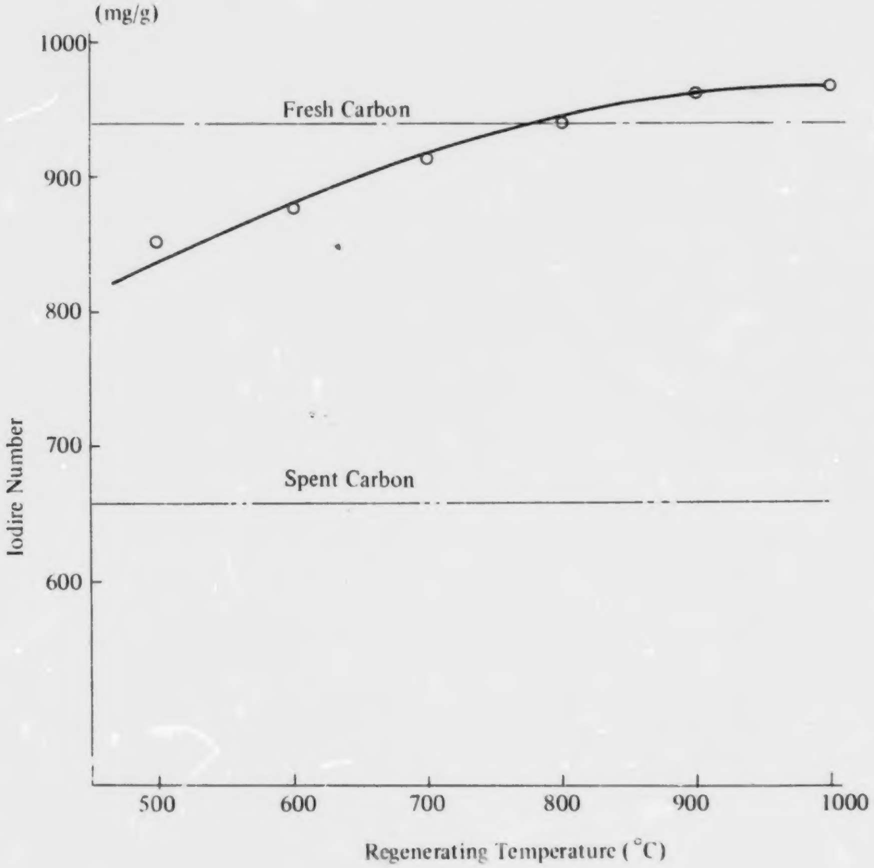


Fig. 6.22 Regenerating Temperature and Iodine Number

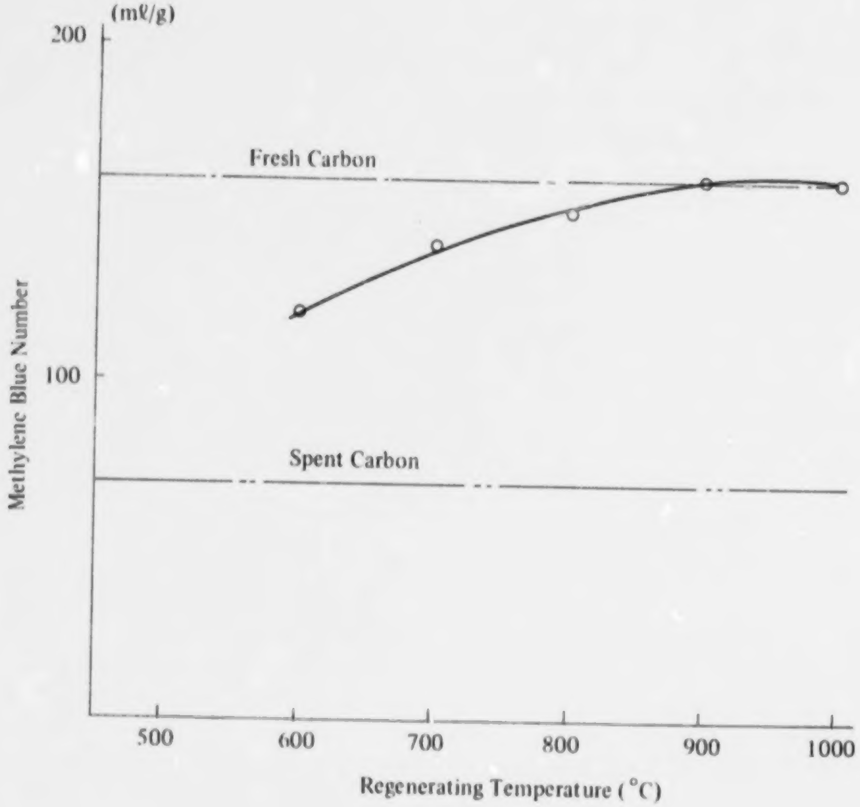


Fig. 6.23 Regenerating Temperature and Methylene Blue Number

CHAPTER 7. AMMONIA REMOVAL BY BREAKPOINT CHLORINATION PROCESS

7.1 Introduction 283

7.2 Laboratory Tests 283
  7.2.1 Breakpoint Chlorination of Secondary Effluent 283
  7.2.2 Effect of pH on Breakpoint Chlorination 284

7.3 Pilot Plant Experiment on Breakpoint Chlorination 285
  7.3.1 Outline of the Pilot Plant 285
  7.3.2 Results of Operation 285
  7.3.3 Troubles over Operation and Performance 286

7.4 Future Research 286

7. AMMONIA REMOVAL BY BREAKPOINT CHLORINATION PROCESS

7.1 INTRODUCTION

The breakpoint chlorination process has been practised as a prechlorination treatment at water treatment. plants receiving water from polluted rivers. The control of prechlorination process for water treatment is usually made by measuring residual chlorine. The breakpoint chlorination process for the removal of ammonia nitrogen in sewage differs slightly from the prechlorination process for water treatment as follows.

1) Ammonia concentration in sewage varies greatly with time.

2) It is desirable to minimize residual chlorine in chlorinated effluent.

3) The effluent to be discharged into receiving waters must not contain residual chlorine.

Now, a pilot plant of breakpoint chlorination process is operated at the Toba Sewage Treatment Plant in Kyoto. This report deals with the results of laboratory tests to determine factors for the pilot plant installations and an outline of the pilot plant.

7.2 LABORATORY TESTS

7.2.1 BREAKPOINT CHLORINATION OF SECONDARY EFFLUENT

The experiments were conducted in a batch system using chlorine water and sodium hypochlorite as the chlorinating agents.

The summary of the results is as follows.

1) Chlorination of the effluent with chlorine water reduces pH, and it is necessary to use alkali for pH elevation. If sodium hydroxide is taken up for pH control, its dose rate should be 1.0 to 1.5 times of the chlorine dose rate, in weight ratio, in order to raise pH up to neutral. Chlorination with sodium hypochlorite does not require pH control.

2) Breakpoint chlorination is greatly affected by pH. For the purpose of optimizing the chlorination, pH must be kept in the range of 7 to 8.

3) The reaction is considered to be completed within 5 min. after chlorine injection. In the event that pH after chlorine dose remains low, the reaction can be promoted further by raising pH elevated near the neutral.

4) Temperature has little effect on reaction.

5) The breakpoint of the secondary effluent is around the 10:1 Wt. ratio of $Cl:NH_3$-N. Cl:N ratios of the breakpoint are essentially the same for both chlorine water and sodium hypochlorite. Hence, it is necessary to dose about 10 mg/l of available chlorine as Cl in order to oxidize 1 mg/l of ammonia nitrogen in the secondary effluent.

6) Nitrate is formed by Chlorination. With the increase in Cl/N ratio, the amount of nitrate increases, but remains within 1 mg/l.

7) The color sometimes develops by chlorine dose above the breakpoint depending upon water quality. This might be ascribable to oxidation of iron or manganese contained in the secondary effluent.

7-2-2 EFFECT OF pH ON BREAKPOINT CHLORINATION

The mechanism of the breakpoint chlorination has not been fully understood. Nevertheless, it is believed that the most important factor affecting the chlorination is pH. Samples used in this senes of experiments were distilled water with ammonium chloride and sodium carbonate. Sodium carbonate was added to the sample to increase buffer capacity. The experiments were conducted in a batch system using sodium hypochlorite as a chlorinating agent. Figs. 7.1 and 7.2 show the results of chlorination with Cl/N ratio adjusted at 8. The initial pH of the buffer solution of ammonium chloride was controlled in the range of 5 to 10.

From these experiments, the following were disclosed.

1) The development of breakpoint in the sample varies with the initial value of pH. Judging from the formation of chloramines, it can be classified into three groups according to initial values of pH; (5,6), (7,8,9) and (10).

2) pH in the sample varied before and after chlorination. The two initial pH groups of (5,6) and (10) showed a rise in pH after chlorination, while the group (7,8,9) showed a drop.

3) The product in the group (5,6) was mainly dichloramine. The formation of monochloramine was not significant. The products from the group (7,8,9) were monochloramine and dichloramine in coexistence in nearly the same concentration. In the group (10), monochloramine alone was present.

It was found that there was a difference in the location of breakpoint among the three groups. Figs. 7.3 and 7.4 show the results obtained by chlorination with Cl/N ratio being changed. The initial pH value of each sample was set at 4.5, 7.5 and 10.2, respectively. The results are summarized below.

1) At pH 10.2, no clear breakpoint was found.
At initial values of 7.5 and 4.5, the breakpoints were around 8 and 9 in terms of Cl/N wt. ratio, respectively. The removals of NH_3-N at the breakpoint were 97% and 93%, respectively.

2) The main product at an initial pH value of 7.5 was monochloramine, but smaller amount of dichloramine was formed. At an initial pH value of 4.5, the main product changed from monochloramine to dichloramine as Cl/N ratio increased. At an initial value of pH 10.2, monochloramine alone was detected.

3) Formation of $(NO_2 + NO_3)$-N after chlorination was barely seen at initial pH of 10.2. At pH 7.5 and 4.5, however, the formation progressed greatly when chlorine dosage exceeded the breakpoint. The concentrations of $(NO_2 + NO_3)$-N at initial values of pH 7.5 and 4.5 were 0.20 mg/l and 0.42 mg/l, respectively at the breakpoint.

4) At initial value of pH 7.5, the pH after chlorination decreased slightly around the breakpoint. At the breakpoint for initial pH 4.5, a sharp decline of pH was noticed during chlorination. Fig. 7.5 shows the chlorination of a ammonium chloride solution without buffer agent. It is evident from the figure that even if the initial pH is around 7, pH reduces considerably unless buffer action is

given.

7.3 PILOT PLANT EXPERIMENT ON BREAKPOINT CHLORINATION

7.3.1 OUTLINE OF THE PILOT PLANT

A pilot treatment facility for the breakpoint chlorination process was installed in the Toba Sewage Treatment Plant, Kyoto, and is now in operation.

Fig. 7.6 shows a flow diagram and control system of the pilot facility. In this facility, ammonia in the sewage is measured by an automatic ammonia analyzer. As a chlorine source, sodium hypochlorite solution containing abrut 15% (W/V) of available chlorine as Cl, which is on market is used. The dose rate of sodium hypochlorite is calculated by the following formula. Dose rate (ml/min) = NH_3-N (mg/l) × inflow rate of sewage (m^3/d.) × Cl/n ratio ÷ (NaOCl concentration (W/V%) × 10^2) ÷ (24 (hrs) × 60 (min)). As aninflow to the process, filtered secondary effluent or effluent from activated carbon contactors is used. The maximum capacity of the facility is designed at 250 m^3/day.

The facility is currently being run with the constant flow rate regulated by valve control. At the maximum design flow rate, the detention time of each chlorination tank is about 5 min. At present, Cl/NH_3-N ratio is manually set. Two automatic ammonia analyzers-colorimetric type and electrode type-are installed in order to investigate their performances as well. Each type has a measuring range of 0 to 20 mg/l. For the control purposes, one of the two analyzers is employed. The automated colorimetric analyzer is divided into two portions; filtering and colorimetering. The filtering portion provides disinfection by heat and filtration of the sample. This analyzing instrument takes roughly 45 min. from sampling to measurement. On the other hand, the electrode type instrument requires about 20 min. for measurement. The dosing pump for sodium hypochlorite controls flow rate by adjusting the stroke and revolutions, and is able to control the dose rate in the range of 30 to 600 ml/min.

A test for treating the breakpoint chlorination effluent by a carbon contactor just started from September of this year.

7.3.2 RESULTS OF OPERATION

Table 7.1 and 7.2 show some of the results of the operation of the pilot plant. Table 7.1 shows the operating conditions and ammonia nitrogen concentration over a period from May 16 to 22. From this table, the followings are found.

1) The average NH_3-N concentrations of the influent and the effluent were 8.42 mg/l and 1.10 mg/l, respectively. Therefore, the average removal of ammonia nitrogen was 86.9%.

2) The concentration of available chlorine in NaOCl solution decreases a little with time.

Table 7.2 shows the results of water quality analysis on May 21 and 22. From table 7.2, the followings are found.

1) The NH_3-N removal was nearly 100%. Considering the fact that the residual chlorine concentration was low, the state was close to the breakpoint.

2) The formations of NO_3-N were 0.56 mg/l and 1.62 mg/l, which were little higher than in the laboratory tests.

3) The removal of COD_{Mn} was about 12.5%.

4) Chloride concentration in the effluent was increased by about 100 mg/l.

7-3-3 TROUBLES OVER THE OPERATION AND PERFORMANCE

The following problems have been brought to the fore since the start-up of the pilot plant.

1) When the ambient temperature rises, sodium hypochlorite decomposes, reducing available chlorine. The decomposition becomes significant when the temperature exceeds 25°C. It was necessary, therefore, to install a dial on the controller to correct the change in NaOCl concentration. The concentration is checked periodically and the dial is adjusted manually.

2) NH_3-N concentration detected by the colorimetric type automatic ammonia analyzer is a little lower than by the manual Nessler's method. This problem is being examined.

3) It has been found that ammonia electrode has a short life. The necessary frequency of calibration is under study.

7-4. FUTURE RESEARCH

As far as the removal of ammonia nitrogen is converned, the breakpoint chlorination is very effective. However, addition of chlorine into liquids like sewage which contains a large amount of organic matter may cause serions problems. The following are left for the future study.

1) Material balance in the reaction between chlorine and ammonia.

2) Removal of chloramines and chlorinated organic compounds by activated carbon adsorption through laboratory and pilot scale studies.

Table 7.1 Results of operators performed 5/16/75 – 5/22/75

Time	Item	Flow rate m3/day	Cl/N Wt. ratio	NaOCl Concentration W/V %	Injection Pump					Influent of Carbon Contactor Colorimetric Analyzer		Note
					Stroke (%)	VS Motor (rpm)	Dose Rate Calc. (ml/min)	Dose Rate Obs. (ml/min)	Obs./Calc.	Influent NH3-N (mg/l)	Effluent NH3-N (mg/l)	
5/16	16 : 00	260	10	14.08				-		9.70	0	Calibration 24 : 00
	20 : 00									10.10	0	
	23 : 00									9.75	0	
5/17	4 : 00	251	10	14.08	measurement 8 : 30					8.45	0	
	8 : 00				65	350	104	100	0.96	8.05	0	
	11 : 00									8.50	0	
	16 : 00									9.60	1.35	
	20 : 00									8.60	1.00	
	24 : 00									7.25	0.70	
5/18	3 : 00	–		–	-	–				6.70	0.80	
	8 : 00									6.10	1.60	
	12 : 00									6.15	2.35	
	16 : 00									7.90	1.20	
	20 : 00									10.20	6.55	
	24 : 00									10.90	4.00	
5/19	4 : 00			–	–	–	–	–	–	10.25	0	
	8 : 00									9.05	3.10	
5/20	16 : 00	255	10	13.86						6.20	0	
	20 : 00									7.00	0	
	24 : 00									7.15	0	
5/21	4 : 00	251	10	13.86	59	370	96	90	0.94	7.10	0	
	8 : 00									7.00	2.00	
	12 : 00									7.35	0	
	16 : 00									8.40	0.55	
	20 : 00									9.25	0	
	24 : 00									9.30	0	
5/22	4 : 00	248	10	13.86	-	-	–	–		9.00	0	
	7 : 00									8.60	0	
	11 : 00									8.35	0	
	16 : 00									9.10	6.20	
	20 : 00									9.95	2.85	

Table 7.2 Results of Tests

Influent of Carbon Contactor

		PH	COD_{Mn} (mg/l)	K-N (mg/l)	NH_3-N (mg/l)		NO_3-N (mg/l)	Cl^- (mg/l)	Cl Residual (mg/l)
					Auto-matic	Manual			
5/21 9 : 00 Cl/N = 10 NaOCl Conc. 13.86% (W/V)	Influent	6.94	11.5	7.46	7.10	6.15	8.00	56.20	–
	Effluent	7.24	10.1	0.43	0	0.03	8.56	166.67	1.60
	Removal	–	12.1	94.2	100	99.5	–	–	–
5/22 9 : 30 Cl/N = 10 NaOCl Conc. 13.86% (W/V)	Influent	7.01	12.6	10.57	8.40	7.25	8.03	58.14	–
	Effluent	6.91	11.0	1.01	0	0.01	9.65	156.98	2.14
	Removal	–	12.7	90.4	100	99.9	–	–	–

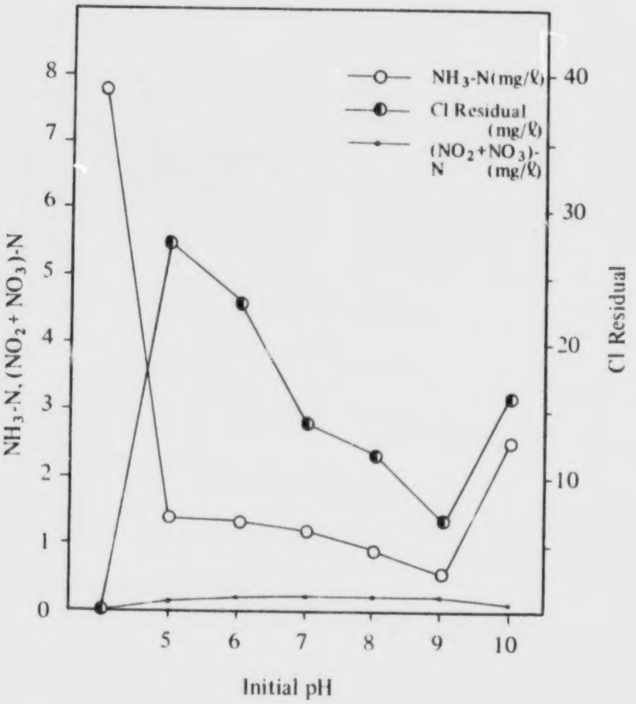


Fig. 7.1 Effect of pH on Chlorination

Cl/N = 8 (wt. ratio), Total alkalinity = 31.5 mg/ℓ

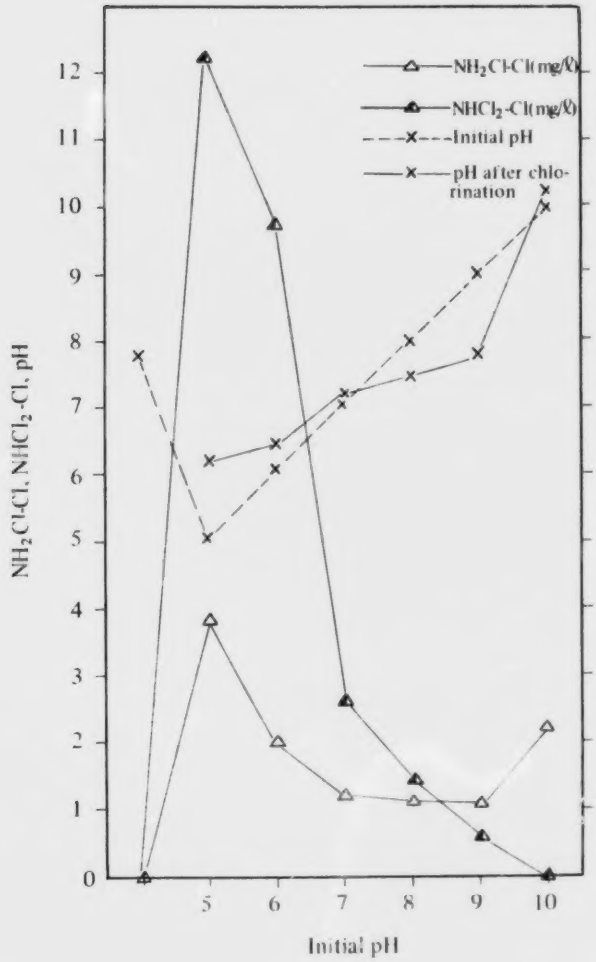


Fig. 7.2 Chloramine Formation

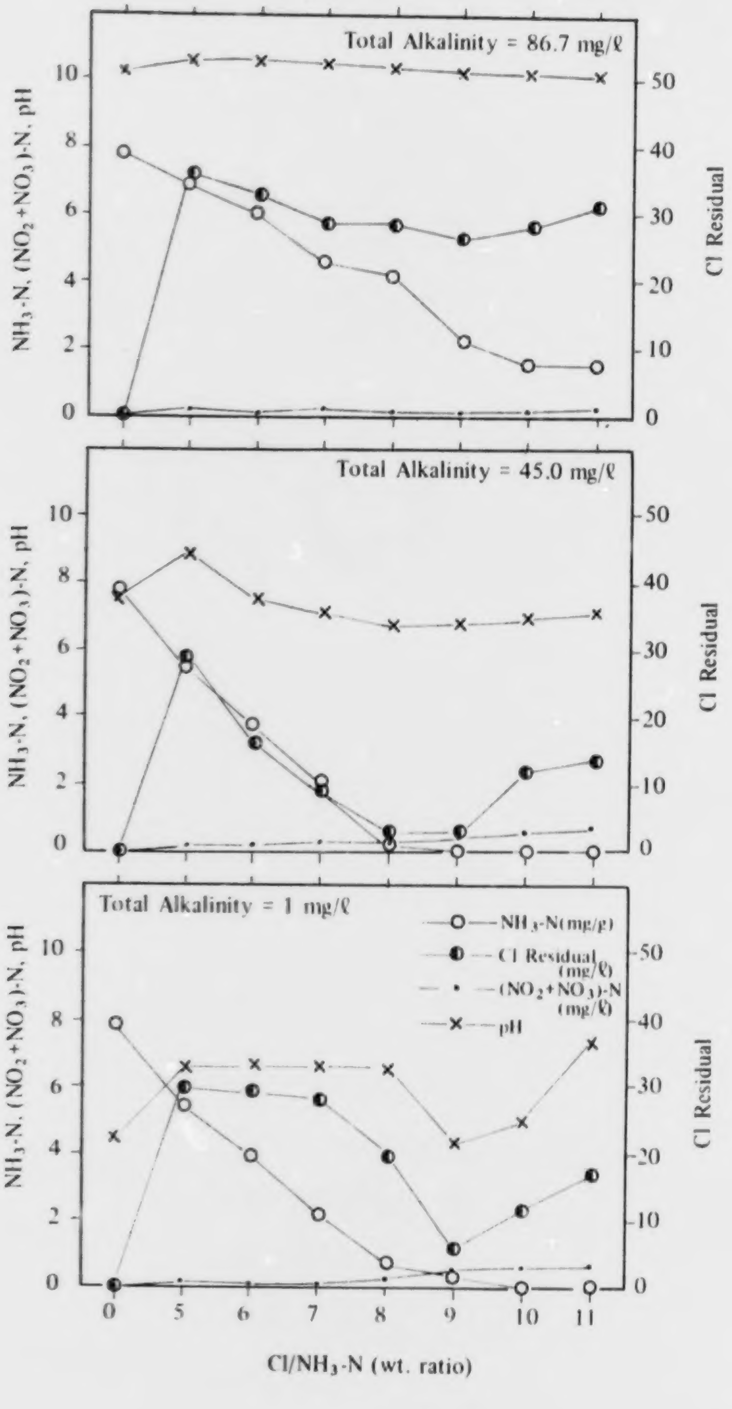


Fig. 7.3 Breakpoint Chlorination and pH

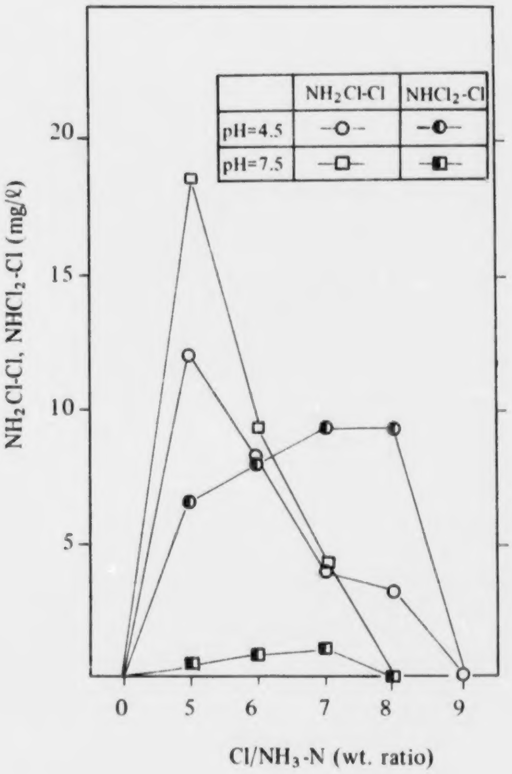


Fig. 7.4 Chloramine Formation

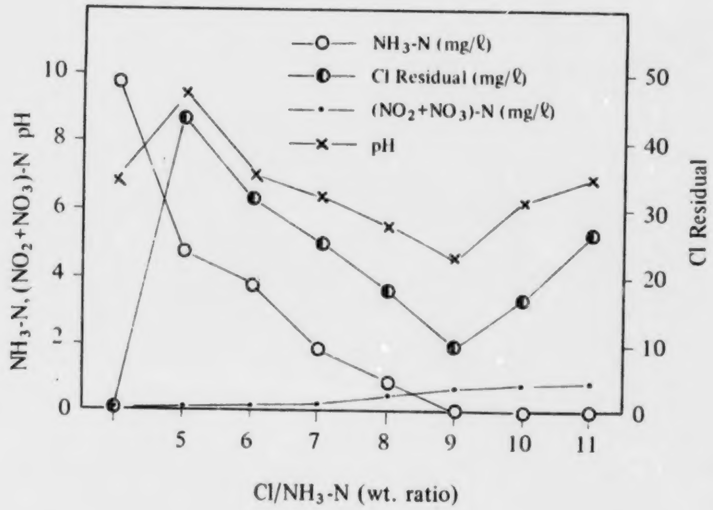
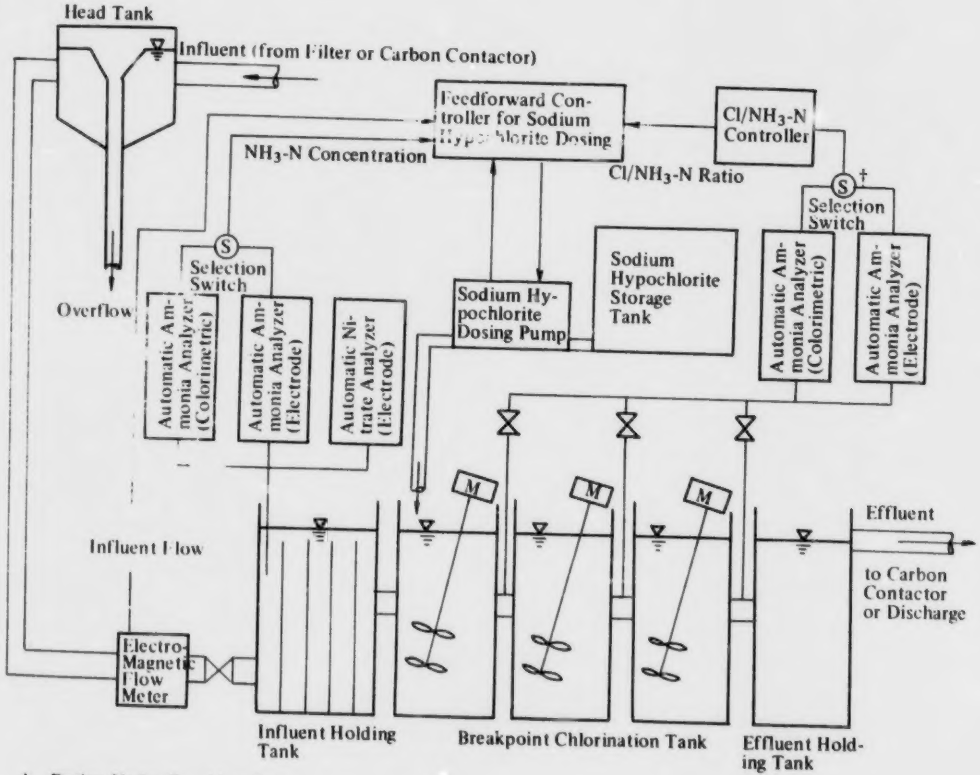


Fig. 7.5 **Breakpoint Chlorination and pH**

Total Alkalinity = 5.5 mg/ℓ


Head Tank
Influent (from Filter or Carbon Contactor)
Feedforward Controller for Sodium Hypochlorite Dosing
Cl/NH3-N Controller
NH3-N Concentration
Cl/NH3-N Ratio
S
Selection Switch
S †
Selection Switch
Overflow
Automatic Ammonia Analyzer (Colorimetric)
Automatic Ammonia Analyzer (Electrode)
Automatic Nitrate Analyzer (Electrode)
Sodium Hypochlorite Dosing Pump
Sodium Hypochlorite Storage Tank
Automatic Ammonia Analyzer (Colorimetric)
Automatic Ammonia Analyzer (Electrode)
M
M
M
Effluent
Influent Flow
to Carbon Contactor or Discharge
Electro-Magnetic Flow Meter
Influent Holding Tank
Breakpoint Chlorination Tank
Effluent Holding Tank

† Design Under Consideration

Fig. 7.6 Flow Chart and Control System of Breakpoint Chlorination

CHAPTER 8. PHOSPHATE REMOVAL IN AN ACTIVATED SLUDGE FACILITY BY ALUM ADDITION

8.1 Outline of Nishiyama Sewage Treatment Plant 296
8.2 Installed Equipment and Devices 297
8.3 Chemical Addition Control 297
8.4 Outline of Methodology 298
8.5 Results and Discussion 298
8.6 Summary 302
8.7 Future Research Project 303

8. PHOSPHATE REMOVAL IN AN ACTIVATED SLUDGE FACILITY BY ALUM ADDITION

Putting newly planned sewage treatment plants aside, the existing sewage treatment plants are said to permit of almost no expansion for functional improvement including phosphorus removal because of difficulties involved in the acquisition of rights-of-way. To ward off this problem a method of dosing coagulants like metal salts, into existing facilities such as aeration tank has been brought into limelight. This is because the expansion work can be limited to a minor extent - to the installation of chemical storage tank, dosing pump and additional sludge treatment facility to overcome excess waste sludge to be developed as a result of chemical dosage.

This method has been evaluated in various ways by E.P.A. of U.S.A. and some countries in Europe, and has been put to practice. In Japan, the raw sewage is usually weak in strength, and the data concerning the treatment of this kind of sewage in this method have been called for.

For example, the domestic sewage in Japan is characterized by BOD of 100 to 180 mg/l. and T-P of 4 to 6 mg/l.

The Public Works Research Institute of the Ministry of Construction and Nagoya Municipal Government formed a joint project for a demonstration survey at Nishiyama Sewage Treatment Plant, which is taking in domestic sewage typical of Japan, of the effects of coagulant on upgrading of the existing plants.

The results of survey will be used not only for Nagoya Municipal Government but also for those cities that are suffering from similar problems in the upgrading of existing sewage treatment facilities. The purposes and objective of the survey to be undertaken by this project are summarized below.

i) Study for a method of reducting the residual phosphorus concentration in effluent below 0.5 mg T-P/lit.

ii) Study on the impact on the entire functions of plant including increased sludge production.

iii) Study on the effects of metal salts on biological treatment.

iv) Study for a method of promoting removal of phosphorus and nitrification simultaneously.

v) Study for a method of removing suspended solids in effluent by rapid sand filtration, polymer addition, etc.

vi) Study for a recovery method of dosed metal salts.

vii) Development of instrumentation and automatic control techniques

viii) Economic assessment

The demonstration project was started in 1974. Now, alum addition into an aeration tank is in the progress. The following is an interim report of the survey for the period from 1974 to July 1975.

8.1 OUTLINE OF NISHIYAMA SEWAGE TREATMENT PLANT

Nishiyama Sewage Treatment Plant is a comparatively small sewage treatment plant for domestic sewage only. The sewage collection is of the separate sewer type, and the conventional plug flow activated sludge system is adopted for biological treatment.

Table 8.1 shows an outline of the design and Table 8.2 an outline of the facilities.

This small-scale domestic sewage treatment plant is always facing vilolent fluctuation in both quality and quantity of influent, and thus has been greatly in need of chemical dose control.

Fig. 8.1 shows diagonal variation of T-P and Sol T-P in the influent.

As shown in Table 8.2, this plant is separated into two bays, and a modification was made to use one bay as a control and the other as a chemical addition train for the purpose of comparing them while running under the same operating conditions.

The modification includes separation of return sludge influent channel and separation of channel leading from the aeration tank to final sedimentation basin. The sludge had been returned from a hopper of the final sedimentation basin via a pump well, and a submerged pump was installed at one bay to permit direct sludge return and withdrawal of excess waste sludge. In this way, the basins were completely separated from each other. But later, seepage, though little, was found between the two.

The flow diagram of the facilities is shown in Fig. 8.2.

8.2 INSTALLED EQUIPMENT AND DEVICES

Installed for the purpose of chemical addition were chemical storage tank (15 m^3), chemical pump, and chemical control equipment.

The pump used was a plunger type one capable of changing both speed and stroke.

Its injection control range was 1 : 30 or 5 to 135 lit./min.

8.3 CHEMICAL ADDITION CONTROL

The chemicals were planned to be dosed to be proportional to phosphorus loadings., measuring phosphrus, by automatic colorimetric analyzer.

At present, however, we are in the process of developing an equipment for automatically measuring the phosphorus concentration in the supernatant of mixed liquor and proportional dosing has not yet been achieved.

For this reason, the delivery rate of the dosing pump was controlled to follow the signal representing the change in influent flow.

The dosing rate was predetermined. Namely, the alum has been dosed to make a constant mole ratio of A/P with respect to daily average phosphorus concentration of the influent.

In this system, if the change in the influent flow takes place in the same way as the change in quality, proportional control of chemical dose can be achieved. As illustrated in Fig. 8.2, the qualitative and quantitative changes do not always take place in the same way. At present, the dosing rate is determined with the daily average phosphorus concentration taken as 4 mg T-P/lit.

The dosing pump motor frequently failed for some time after start-up of the project.

It was then found that a sharp decline in the influent flow into the aeration tank due to withdrawal of sludge from the primary sedimentation basin outpaced the dosing pump. To overcome this problem, the motor speed was reduced when the influent flow decreased sharply. Now the dosing pump is well in service.

8.4 OUTLINE OF METHODOLOGY

Coagulant and its addition

The coagulant used was liquid aluminum sulfate (alum). Its chemical formula is $Al_2(SO_4)_3 \cdot 18H_2O$, and Al_2O_3 accounts for some 8%.

Dosing rate was set at 2 in terms of mole ratio of Al/P to T-P in influent or 8 mg/lit. in Al. The dosing position of alum was at the end of the aeration tank.

Later it was found that around this position T-P in the supernatant of mixed liquor is in many cases about 60% of that in the influent, and the dosing rate now is reduced half.

Sampling procedure

Samples were taken by an automatic sampler at an interval of 2 hrs to make up a 24-hr composite sample. The analysis was made 3 to 4 days a week.

Measuremet of sludge

For both bays, the waste sludge was measured with an electromagnetic flowmeter and an ultrasonic solid meter automatically.

8.5 RESULTS AND DISCUSSION

Addition of alum into the end of the aeration tank started in early February, 1975. The dosing rate remained the same until July, but the period was divided into two: Phase I from the start to March 30 when water temperature was low and Phase II from mid-April to the end of July when water temperature rose above 17°C.

Table 8.3 shows the characteristics of the qualities of influent and effluent.

Table 8.4 shows the summary of operating conditions.

In Phase II, the influent flow was a little larger than in Phase I; so was MLSS.

Phosphorus removal

Removal of phosphorus was drastically improved by the addition of alum. As seen in the table, both Phase I and Phase II saw a reduction of residual phosphorus concentration in effluent to 0.4 mgP/lit. in T-P and 0.3 mgP/lit. in Orth-P.

In both phases, the phosphorus concentration in the influent was around 3.5 mg/lit., and the removal efficiency was kept of about 90%.

Up until now, the operation has been continued for about six consecutive months with a Al/P mole ratio set at 2. During this entire period, T-P in the effluent has always been kept below 0.5 mg/lit., proving that so far as this dosing rate is used, the target residual phosphorus concentration of less than 0.5 mgP/lit. can be attained.

In the demonstration palnt, the control bay also showed a comparatively high phosphorus removal of 45 to 55% or less than 2 mgP/lit. in terms of residual phosphorus concentration.

Now investigations are under way as to whether this is due to biological uptake or coagulation by cations like Ca^{++} entrained with influent.

In Phase I, the phosphorus content in the activated sludge in the control bay was 1.7 to 1.9%.

Nitrogen removal and nitrofication

In Phase I when water temperature was low, neither dosed bay nor control bay had nitrification. But in Phase II, the control bay alone experienced substantial nitrification as water temperature rose up. In early July, the nitrification percentage based on TKN reduction reached no less than about 60%. The transition is shown in Fig. 8.3. Alum addition bay did not have nitrified effluent at all for all that it was operated under the same conditions except for dosing. This may be due to depletion of alkalinity or interference by aluminum.

In the dosed bay, washout of suspended solids took place extremely in Phase II, and SRT was as low as about 2 days. This may also be attributable to the prevention of nitrification. As shown in Table 8.3, the influent alkalinity (85 mg/lit.) was consumed 45 mg/lit. as $CaCO_3$ in the dosed bay by the addition of alum, manifesting the depletion of alkalinity necessary for nitrification.

In the chemical bay, aluminum of some 8 mg/lit. was dosed. The accumulation in the sludge reached about 12% in Phase II. Now laboratory test is under way as to how this concentration of aluminum can interfere with the nitrification.

As shown in Table 8.4, the settler has a large overlfow rate, and solids are liable to be washed out. Accordingly, MLSS in tank cannot be maintained high enough whereby SRT cannot be maintained. To cope with this problem, a method of reducing the overflow rate is now under way.

In winter this year, experiments will be made with consideration given to the addition of alkalinity, intereference of Al^{+++}, and increase of SRT in hopes of providing something of a basis for the promotion of nitrification in the process.

Removal of organic matter

Removal of both BOD_5 and TOC was increased by the addition of alum. Compared with the control bay, the dosed bay showed an increase of some 5% in removal of BOD and some 10% in removal of TOC.

In Phase II, the removal slightly went down in the dosed bay, which might be attributable to washout of solids.

Soluble BOD in the effluent was 3.6 mg/lit. and soluble TOC 17 mg/lit., or 55% and 82% to total, respectively.

Solid removal

Phase I and Phase II showed a sharp contrast in the removal of suspended solids.

In Phase I, the residual solids in the effluent was smaller in the dosed bay than the control bay, whereas in Phase II, the control bay showed a smaller value than the dosed bay.

This might have been due to the following causes.

1) Washout of floc produced by aluminum dose after being transformed into

particulates.

In Phase I, data covered comparatively earlier stage of dose though taken after a cell residence time, and washout of fine floc was not noticed.

Aluminum content in the solids in the dosed bay was 7% in Phase I and more than 10% in Phase II.

2) In Phase II, the overflow rate in the control bay was half as much as that in the dosed bay.

Washout was observed in the morning when influent flow was large.

As shown in Table 8.3, the turbidity also showed the same tendency as SS.

From Oct. this year, a rapid sand filter plant will start operation in order to launch into a project for solid removal from the effluent.

pH reduction and alkalinity consumption

pH reduction was not so serious as expected. pH reduction due to addition of alum is about 1 unit, and pH at the end of aeration tank immediately after dose was about 6.1.

Alkalinity is consumed by chemical dose. Addition of 1 mg/lit. of Al^{+++} resulted in consumption of alkalinity by 5.5 mg/lit. max. as $CaCO_3$. In the dosed bay, the consumption was 40 to 45 mg/lit. which corresponded to 8 mg/lit. of added Al^{+++} considering the portion used for coagulation of phosphorus. In Phase II, dosed bay saw an alkalinity consumption of some 30 mg/lit. which may have been due to nitrification.

Alkalinity of influent running into Nishiyama Sewage Treatment Plant is 80 to 120 mg/lit. For perfect nitrification or in the vent of dosing when the phosphorus concentration is high, addition of alkalinity will be almost indispensible.

This may be a matter common to all the sewage treatment plant in Japan which are processing domestic sewage.

Effects on biological activity and microfauna

The survey of the effects of high-concentration aluminum dose to aeration tank on microfauna in the activated sludge was one of the greatest interests attached to the project. Some reports of similar surveys claim that such dose have little effect on the microfauna. In the project, no particular effects were found so long as the effluent quality was concerned, though VSS decreased from 70% to 60%.

Table 8.5 shows the results of microscopic observation of population number of microfauna in the activated sludge in both bays in Phase II.

The differences between the two bays were as follows.

1) Alum dose decreases the number of organisms in the activated sludge on the whole.

2) Number of species of microfauna in the activated sludge is decreased. Fig. 8.4 shows time-dependent change of the number of individuals of ciliata per unit MLVSS.

In Phase II, the effluent of dosed bay became milk, white sometimes after rainfall. At first, it was considered attributable to the promotion of washout of fine floc of aluminum hydroxide. Later, it was found by a microscopic observation to be swarms of bacteria analogous to spirochaeta plicatilis.

The bacteria oftentimes were found even in fine weather, though they were little in number.

The reason why they grow so far as to turn the effluent milk white is still unknown. Whether the bacteria is spirochaeta plicatilis or not is under study.

Sludge production

The sludge production rate is compared in Table 8.6 between the two bays over the 11-day period during which aluminum concentration in the mixed liquor was considered stable.

As touched upon in the foregoing, a little seepage was noticed between the mixed liquors of both bays.

In Table 8.6, the seepage is compensated for with aluminum addition in the control bay as a basis. As shown in Table 8.6, the sludge production in the alum-dosed bay increased as compared with the control bay. The daily average sludge production rate over the 11 day period was 613 kg/d. (45.4 mg/lit.) in the control bay as against 856 kg/d. (72.5 mg/lit.) in the alum dosed bay. Namely, alum dose created an increase of some 30%. During this period, BOD_5 removal was 590 kg/d. (56 mg/lit.) in the control bay as against 640 kg/d. (61 mg/lit.) in the alum dosed bay, with a difference of 50 kg/d. (5 mg/lit.) between the two. It is evident that production of aluminum phosphate and aluminum hydroxide due to alum addition has a direct influence over the increase of sludge production.

Assuming that precipitation reaction of alum follows the following formula, and that 3 mg/lit. of phosphorus in the influent is precipitated with Al^{+++}, the production of sludge due to alum addition will be 184 kg/d. (15.6 mg/lit.) in the form of aluminum hydroxide and 139 kg/d. (11.8 mg/lit.) in the form of aluminum phosphate.

$$Al_2(SO_4)_3 + 6HCO_3 \rightarrow 2Al(OH)_3 + 3SO_4^{-2} + 6CO_2$$

$$Al_2(SO_4)_3 + 2PO_4^{-3} \rightarrow 2AlPO_4 + 3SO_4^{-2}$$

Namely, Aluminum in the sludge in this case will amount to about 110 kg/d or some 12% of the total sludge volume of 856 kg/d. This estimate showed relatively high agreement with the observed value.

It was very interesting to notice during the period that the increment in the alum dosed system washed out along with the effluent.

As regards the sludge production, data collection will be made from a long-term standpoint.

Dewaterability of the chemical sludge

Dewaterability of alum-treated sludge was examined. For the time being, a filter leaf test modeling after vacuum filtration is under way.

Interim results may be summarized as follows.

1) For the dewatering of waste sludge developed by alum addition, dose of calcium hydroxide and ferric chloride in combination as conditioner is effective.

2) For sludge with solid content of 2.5% and aluminum content of 10%, filtration

velocity can be 10 $kg/m^2/hr$ when calcium hydroxide is dosed 50% and $FeCl_3$ 5%. The moisture content of cake in this case is about 85%.

3) There is little difference in dewaterability of waste activated sludge between control bay and dosed bay.

4) Dewaterability can be improved sharply by adjusting the ratio of chemical sludge to primary sludge.

The above refers to the dewatering of raw sludge. In Japan, the dewatering is practised mostly after anaerobic digestion, and study will be made on the dewaterability of sludge containing metal salts after anaerobic digestion.

Cost of chemicals

If the sewage is to be treated with Al/P mole ratio of 2, its treatment cost is estimated at ¥1.78/m^3 (2.32 ¢/1,000 gal.) based on the cost of liquid alum purchased by the Nagoya City.

The sewage considered here contains an average phosphorus concentration of 3.5 mg/lit., and the residual phosphorus concentration in the effluent is less than 0.4 mg/lit. If the phosphorus concentration removed is 3 mg/lit., the chemical cost per removal concentration is estimated to be ¥0.59/m^3/mgP/lit. removed (0.774 ¢ 1,000 gal./mgP removed/lit.).

These are 40 to 50% higher than those appearing in U.S. EPA Seatle's report, EPA Project No. WRRD 247-01, and the cost for removal of phosphorus by 1 mg/lit. is 2.5 times as much.

In the Seatle's experience the average residual phosphorus concentration in the effluent was as high as 2.1 mg/lit., and hence the chemical could be used effectively.

In Japan, aluminum entirely counts on imports, and the alum cost is high, sending up the cost of treatment.

The cost of purchase by the Nagoya City of liquid alum (Al_2O_3, 8%) is ¥11,000/Al_2O_3 ton (\$38/Al_2O_3 ton) or twice as much as that in the United States.

The impact of the reduction of dosing rate on the purchase cost and treatment cost as a whole will be examined.

8.6 SUMMARY

Alum was dosed at the end of the aeration tank in order that Al/P mole ratio to the phosphorus concentration in influent be maintained at 2.

As a result of alum addition over a period of some six consecutive months, the following are summarized.

With reference to raw influent with BOD of 100 mg/lit., and alkalinity of 90 mg/lit. as $CaCO_3$,

1) the removal of phosphorus could be maintained above 90% with the result that the residual phosphorus concentration in the effluent could be reduced below 0.5 mg/lit.;

2) the removal of organic matter increased 10% for TOC and 5% for BOD compared with the control.

3) alum-dosed bay did not have nitrification, demanding the necessity of study on

the addition of alkalinity and effect of aluminum;

4) the microfauna in the dosed bay became different from that in the control bay, showing extermination or decrease of population and species;

5) the sludge production increased some 30% to 40% from that in the control bay, increment being attributable to the addition of aluminum;

6) organic matter in MLSS was reduced in alum dosed system to 60% as against 70% of the control system.

7) keeping MLSS at a high level was difficult, mainly because of high overflow rate observed in the final sedimentation tank, and as a result, SS in the effluent was increased compared with the control;

8) no trouble was developed in the treatment by the low MLSS, suggesting that running at a low MLSS may be practicable where the influent is weak as with the sewage in Japan;

9) the alum dosing cost was ¥1.78/m^3 (2.32 c/1,000 gal.). An experiment with a decreased dosing rate is under way, and precise cost analysis will be made soon.

8.7 FUTURE RESEARCH PROJECT

1) Determination of minimum alum dosing rate necessary to keep the residual phosphorus concentration in the effluent below 0.5 mgP/lit.

2) Study on the promotion of nitrification when alum dosing is practised.

3) Clarification of sludge production balance.

4) Establishment of proportional chemical dosing control system.

5) Feasibility study on the recovery of slum.

6) Study for a method of removing solids in the effluent

At present, a rapid sand filter pilot plant having an effective surface area of 1 m^2 is under construction at Nishiyama Sewage Treatment Plant. Its operation is scheduled for early Oct. this year.

This filter is divided into 8 basins, and when on basin is being subjected to backwashing, the effluent of the other basins is used as backwash water.

This is accomplished automatically according to the total head loss increased. The transfer of backwash water is undertaken by siphon effect, dispensing with valves otherwise required.

This automatic backwashing, valveless rapid sand filter will be used at once for the testing of solid removal from the secondary effluent and for the collection of basic design data necessary for engineering the application of this kind of filter to the solid removal from the secondary effluent.

7) Examination of merits and demerits of dosing coagulants into primary sedimentation basin.

8) Dosing of iron salts and the like other than alum.

9) Economic appraisal

Table 8-1 Outline of Nishiyama STP

Area served	Population served	Volume of sewage flow		Collection System
		Daily average	Daily maximum	
748 (ha) 1,850 (acre)	46,000	20,000 (m^3/d) 5.7 (mgd)	30,000 (m^3/d) 7.9 (mgd)	Separate Sewer

Table 8-2 Outline of Nishiyama Facilities

Item / Facility	Type	Dimension (m)	Number	Total volume (m^3)	Design detention time
Grit chamber	Rectangular	W × L × H 2.5 × 10 × 1.85	2	–	1.2 min.
Preparation tank	Diffused aeration	4.0 × 20 × 3.5	1	245	1.3 min.
Primary settler	Rectangular	5.0 × 28 × 30	4	1,680	1.3 hr.
Aerator	Diffused aeration	5.0 × 40 × 50 × 2 bay	2	4,000	3.2 hr.
Final settler	2 storied rectangular	5.0 × 1st 22.5 / 2nd 27.5 × 3.0	3	2,250	1.8 hr.

Notes: 1. Influent coming in by gravity.

2. Sludge is being treated at the adjacent plant.

Table 8-3 Summary of Influent and Effluent Quality (Phase I)

Feb. 25 ~ Mar. 26, 1975

Item / Category	Raw sewage		Primary effluent		Secondary effluent (Control)		Secondary effluent (Alum. addition)		Removal efficiency over secondary	
	Average	Range	Average	Range	Average	Range	Average	Range	Control	Alum. add.
Temp (°C)	–	11.6 ~ 13.6	–	11.8 ~ 13.6	–	11.7 ~ 13.6	–	11.7 ~ 13.6	–	–
pH	–	6.95 ~ 7.50	–	6.76 ~ 7.60	–	6.94 ~ 7.70	–	6.35 ~ 7.30	–	–
Transparancy	–	2.0 ~ 4.5	–	3.0 ~ 8.0	–	15.2 ~ 30 <	–	2.4 ~ 3.0 <	–	–
Turbidity	130	67 ~ 223	83	51 ~ 111	14	26 ~ 65	7	1 ~ 25	83.1	91.6
SS (mg/ℓ)	135	60 ~ 232	52	34 ~ 77	12	2 ~ 36	6	2 ~ 14	76.9	88.5
Total Alkanility	94	79 ~ 118	92	69 ~ 112	87	70 ~ 103	52	43 ~ 65	21.7	43.6
BOD_5 (mg/ℓ)	100	96 ~ 104	67	51 ~ 83	8	9 ~ 16	5.3	5.0 ~ 5.5	88.0	92.1
COD (mg/ℓ)	69	54 ~ 110	48	42 ~ 61	14	12 ~ 17	10	9 ~ 11	70.8	79.1
TOC (mg/ℓ)	78	77 ~ 80	69	67 ~ 72	40	38 ~ 42	17	11 ~ 24	42.0	75.3
T-P (mg/ℓ)	3.44	2.80 ~ 4.40	3.44	29.3 ~ 4.0	1.95	1.3 ~ 2.7	0.27	0.2 ~ 0.4	43.3	92.2
Orth-P (mg/ℓ)	2.95	1.5 ~ 6.56	2.25	1.71 ~ 3.33	1.42	0.91 ~ 1.67	0.13	0.08 ~ 0.28	36.7	94.2
TKN (mg/ℓ)	22.8	22.5 ~ 23.0	23.7	22.4 ~ 25.0	18.8	18.6 ~ 19.0	17.4	16.9 ~ 17.8	20.7	26.5
NH_3-N (mg/ℓ)	12.2	11.3 ~ 13.1	13.9	13.2 ~ 14.6	14.1	14.0 ~ 14.2	14.4	14.0 ~ 14.7	–	–
NO_2+NO_3-N (mg/ℓ)	0.4	0.1 ~ 0.3	0.2	0.1 ~ 0.3	0.3	0.28 ~ 0.32	0.3	0.15 ~ 0.34	–	–

Table 8-3 Summary of Influent and Effluent Quality (Phase II)

April 12 ~ June 30, 1975

Item / Category	Raw Sewage		Primary effluent		Secondary effluent (Control)		Secondary effluent (Alum. addition)		Removal efficiency over secondary	
	Average	Range	Average	Range	Average	Range	Average	Range	Control	Alum. add.
Temp (°C)	–	16.0 ~ 21.8	–	16.0 ~ 21.8	–	16.4 ~ 21.8	–	16.4 ~ 21.8	–	–
pH	–	6.8 ~ 7.2	–	6.8 ~ 7.2	–	6.6 ~ 7.2	–	6.4 ~ 7.3	–	–
Transparancy (cm)	–	2.5 ~ 5.5	–	3.0 ~ 7.0	–	1.7 ~ 30	–	6.5 ~ 30	–	–
Turbidity (mg/ℓ)	80.3	56.5 ~ 132	52.02	32.5 ~ 92	6.7	4.1 ~ 11.3	15.7	3.5 ~ 62	87.1	69.9
SS (mg/ℓ)	118	81 ~ 162	41	31 ~ 78	6	2 ~ 11	21	2.0 ~ 66	84.5	49.7
Total Alkalinity (mg/ℓ, $CaCO_3$)	80.5	59 ~ 126	85.3	62 ~ 135	55	30 ~ 95	39	13.5 ~ 53.5	–	–
BOD_5 (mg/ℓ)	98.7	68.8 ~ 124	64	47 ~ 95	9.5	6.5 ~ 16.5	6.4	4.7 ~ 8.3	85.1	90.0
COD (mg/ℓ)	56.1	36.4 ~ 76.4	41.4	32.3 ~ 55.7	11.3	8.3 ~ 14.5	12.5	7.0 ~ 37.5	72.7	69.8
TOC (mg/ℓ)	80.1	51.8 ~ 122	58.8	38.4 ~ 87.0	19.3	8.1 ~ 36.0	14.9	7.1 ~ 22.3	67.1	74.6
T-P (mg/ℓ)	3.54	2.59 ~ 4.92	3.45	2.01 ~ 7.90	1.49	0.67 ~ 3.21	0.39	0.14 ~ 0.69	56.8	88.7
Orth-P (mg/ℓ)	2.16	0.81 ~ 3.20	2.24	0.79 ~ 4.40	1.25	0.59 ~ 1.97	0.31	0.06 ~ 0.75	44.2	86.2
TKN (mg/ℓ)	25.2	21.8 ~ 29.1	25.8	20.6 ~ 39.1	13.2	9.9 ~ 21.0	17.8	16.0 ~ 21.6	49.1	31.3
NH_3-N (mg/ℓ)	14.3	12.1 ~ 16.8	15.7	12.2 ~ 25.5	9.7	6.6 ~ 17.4	14.1	12.3 ~ 16.4	38.3	10.3
NO_2+NO_3-N (mg/ℓ)	0.11	0.0 ~ 0.28	0.04	0 ~ 0.10	3.17	0.20 ~ 6.7	0.35	0.07 ~ 0.83	–	–

Table 8-4 Summary of Plant Operation

Phase	Category \ Item	Average daily flow (m^3/d)	Aerator							Final settler	
			Aeration time (h)	Return sludge rate (%)	Air flow rate (times)	MLSS (mg/ℓ)	MLVSS (mg/ℓ)	SVI	Organic load (kg BOD/kg SS)	Detention time (hr)	Overflow rate ($m^3/d/m^3$)
Phase I	Control	10,650	4.5	30	4	640	460	117	0.55	1.7	42.6
	Alum. addition	10,650	4.5	30	4	950	645	115	0.37	1.7	42.6
Phase II	Control	11,800	4.1	30	4	1,030	717	105	0.36	3.1	24
	Alum. addition	11,800	4.1	30	4	930	560	111	0.40	1.5	47.2

Table 8-5 Microfauna in Activated Sludge (Alum. Addition)

(N/ml)

Species \ Data	May 20	May 22	May 23	May 28	June 2	June 4	June 5	June 9	June 12	July 2
Euglena	–	–	200	–	–	–	–	–	–	–
Unknown flagellata	60	180	40	–	–	–	–	–	–	–
Tetrahymena	40	20	20	–	–	20	–	–	–	–
Litonotus	–	–	–	–	–	–	–	–	–	–
Unknown ciliata	–	–	–	–	–	–	–	–	–	–
Aspidisca	–	20	80	–	20	20	40	–	–	40
Vorticella	60	20	100	100	240	720	200	40	200	120
Opercularia	–	–	–	–	–	–	–	–	–	–
Carchesium	–	40	–	–	80	80	–	–	–	–
Zoothamnium	–	–	–	–	–	–	–	–	–	–
Epistylis	80	80	120	140	40	160	20	–	140	200
Rhabdostyla						100				
Tokophrya	20	40	140	–	–	60	–	80	140	220
Rotaria	–	–	20	–	–	–	40	20	40	–
Nematoda	–	20	20	20	40	20	–	–	–	20

Table 8-5 Microfauna in Activated Sludge (Control)

(N/ml)

	May 20	May 22	May 23	May 28	June 2	June 4	June 5	June 9	June 12	July 2
Euglena	40	–	420	–	–	–	–	–	–	–
Unknown Flagellata	–	80	730	+++	+++	+++	+++	+++	100	+++
Tetrahymena	260	320	2,000	260	580	120	–	86	–	2,340
Litonotus	60	–	380	720	–	–	–	–	–	–
Unknown ciliata					160					
Aspidisca	620	340	780	300	760	160	–	500	500	640
Votticella	–	80	160	720	500	2,040	260	2,080	980	820
Opercularia	–	–	–	–	–	–	–	–	–	20
Carchesium	–	–	–	–	100	200	–	220	360	–
Zoothamium	–	220	–	–	–	–	–	80	–	40
Epistylis	640	80	160	1,080	2,160	3,260	4,050	1,080	40	200
Rhabdostyla										
Tohophrya	–	100	240	180	120	300	60	160	100	20
Rotaria	20	20	–	–	60	20	20	200	120	320
Nematoda	–	20	–	–	–	–	–	20	40	40

Table 8-6 Comparison of Sludge Production

Sludges	Control	Alum. addition
Waste activated suldge		
kg/d.	536	545
(mg/ℓ)	(45.4)	(46.1)
Solid in effluent		
kg/d.	77	310
(mg/ℓ)**	(6.5)	(26.2)
Total		
kg/d.	613	856
(mg/ℓ)**	(45.4)	(72.5)
Addition as $Al(OH)_3$*)		
kg/d.	–	184
(mg/ℓ)**	–	(15.6)
Addition as $AlPO_4$*)		
kg/d.	–	139
(mg/ℓ)**		(11.8)

*) Calculated value. **) Based on inflow.

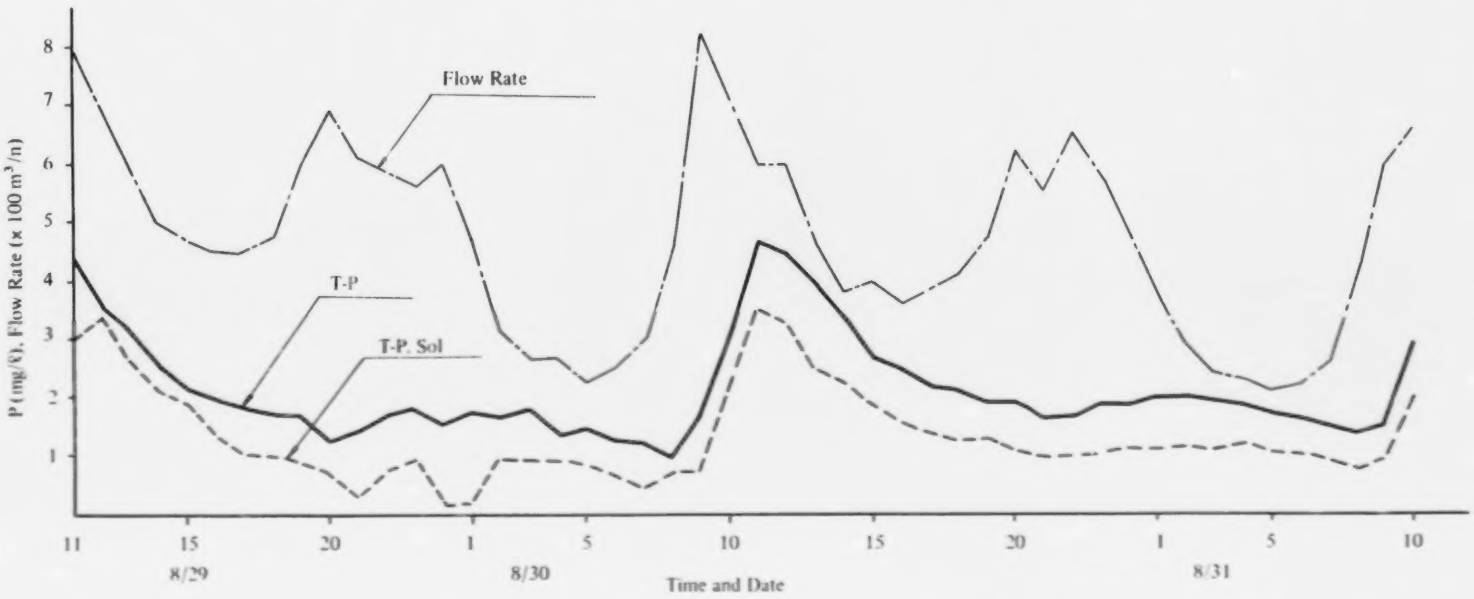


Fig. 8.1 Fluctuation of Flow Rate and Phosphorus Concentration in Primary Effluent

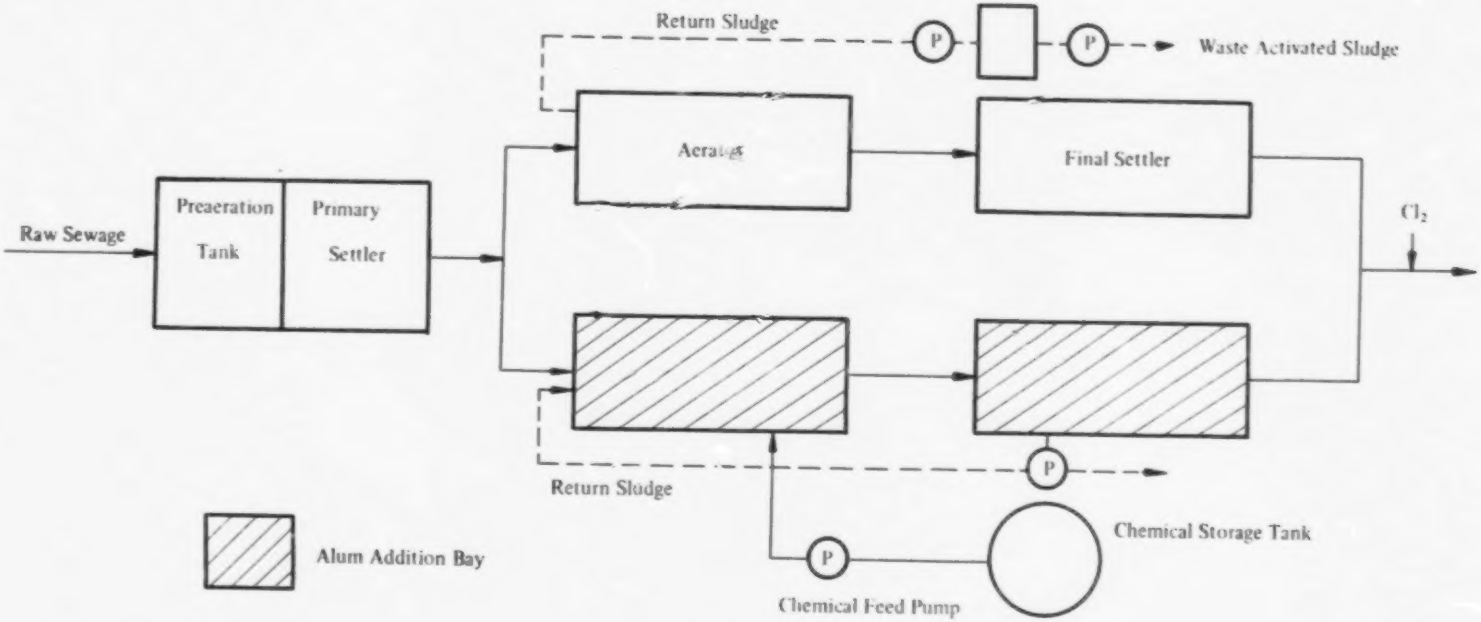

Return Sludge
P
P
Waste Activated Sludge
Raw Sewage
Preaeration Tank
Primary Settler
Aeration
Final Settler
Cl_2
P
Return Sludge
Alum Addition Bay
P
Chemical Storage Tank
Chemical Feed Pump

Fig. 8.2 Flow Diagram of Nishiyama STP

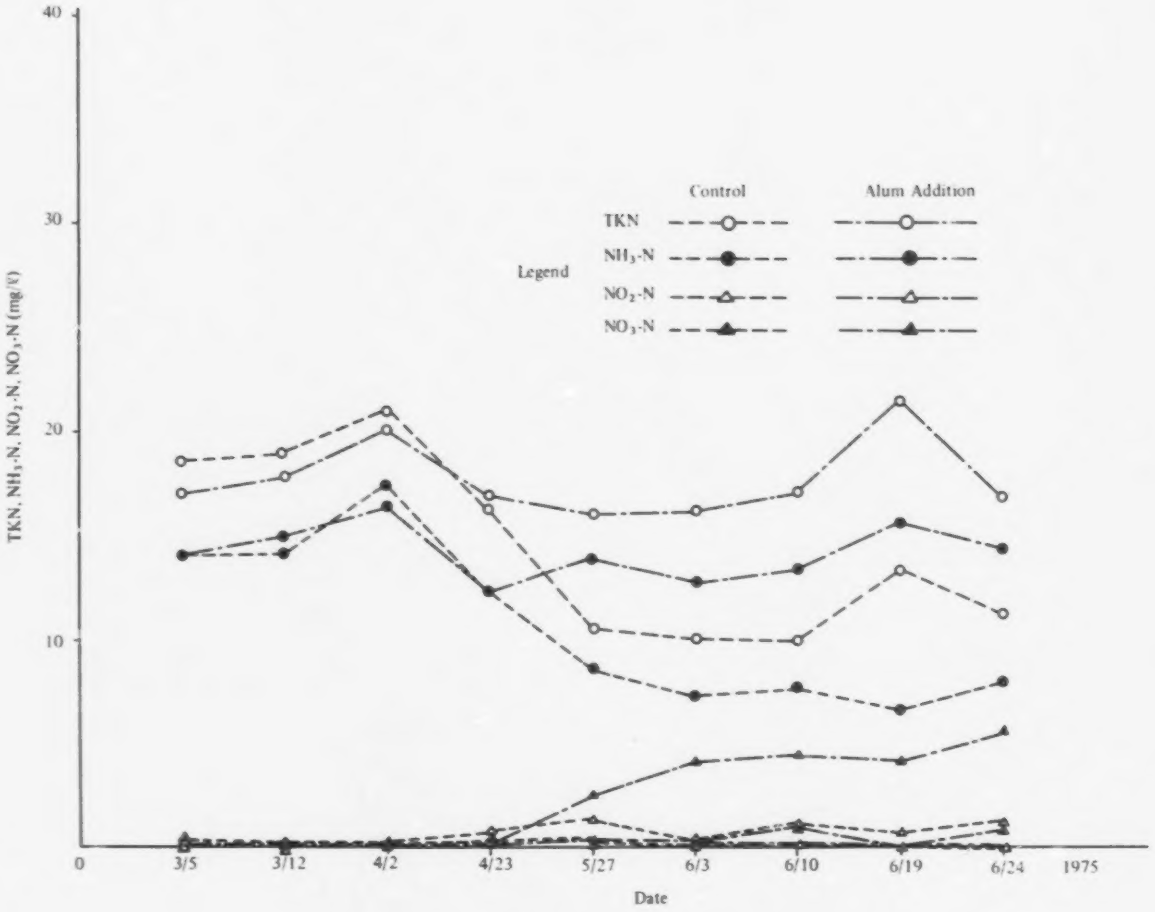


Fig. 8.3 Weekly Change of Nitrogen in Final Effluent

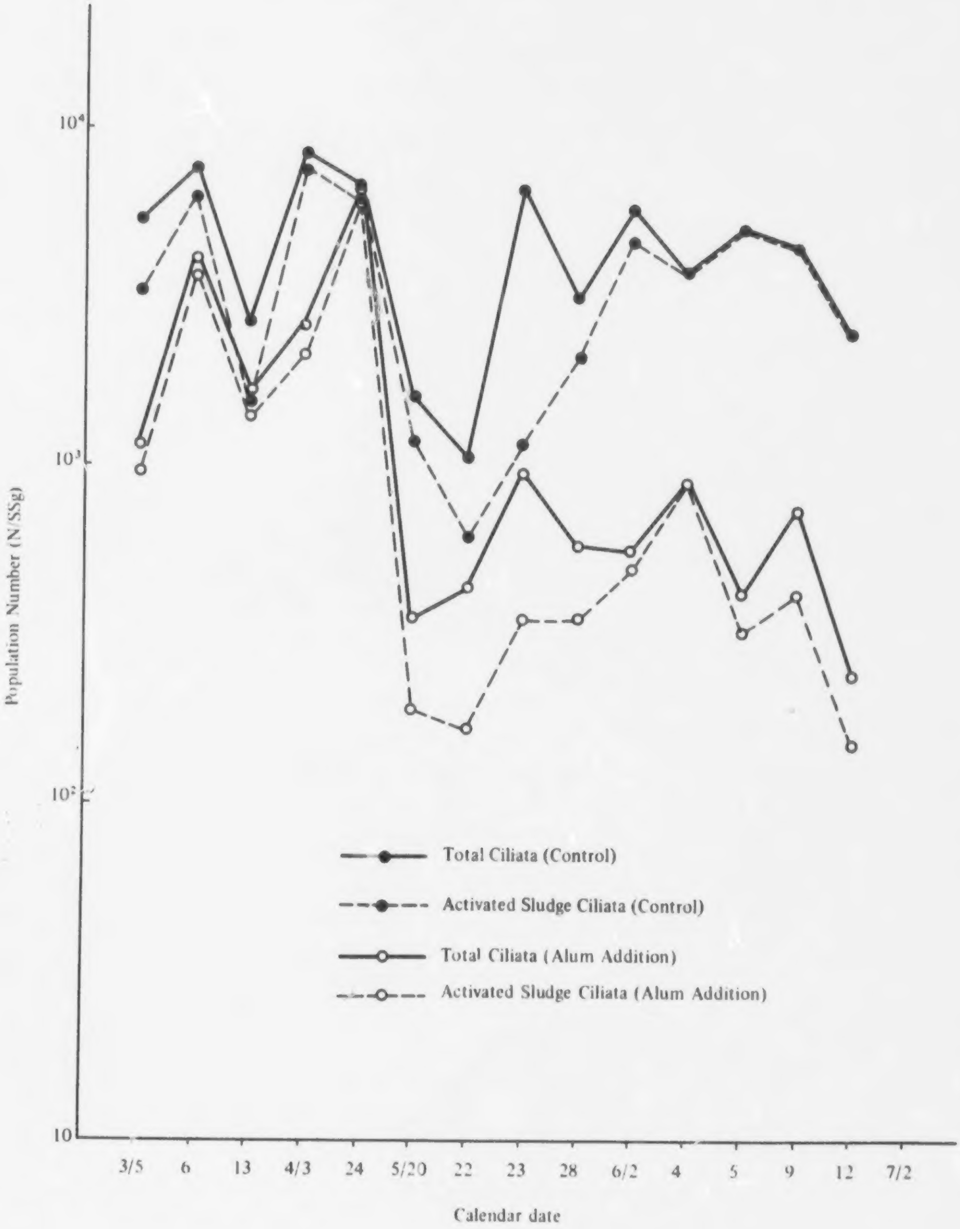


Fig. 8.4 Changes in Microfauna in the Activated Sludge

CHAPTER 9. PILOT PLANT STUDIES OF PHOSPHORUS REMOVAL FROM SECONDARY EFFLUENT TO PROTECT LAKE BIWA

9.1 Lake Biwa 317

9.2 Present Situation of Water Quality at Lake Biwa 321

9.3 Eutrophication Control and Effluent Quality Standard 323

9.4 Removal of Phosphorus 327
  9.4.1 Back Ground 327
  9.4.2 Pilot Plant for Phosphorus Removal 327
  9.4.3 Phosphorus Removal 330

9.5 Sludge Handling 330
  9.5.1 Alum Sludge 330
  9.5.2 Thickening 331
  9.5.3 Dewatering 331

9.6 Further Research Needs 334
  9.6.1 Loading of Phosphorus and Nitrogen on Lake Biwa 334
  9.6.2 Further Pilot Plant Study 335

9.1 LAKE BIWA

The Biwa is one of the largest lake in Japan; 680 km² in area, and it occupies one sixth portions of total Shiga Prefecture area.

Almost all the area of Shiga Prefecture is the basin at Lake Biwa.

Yodo-River rise from Lake Biwa is the main water resource for Kyoto, Osaka, and Kobe. These cities are located in one of the two most densely populated and industrialized areas, Tokyo and Kansai metropolitan areas, in Japan.

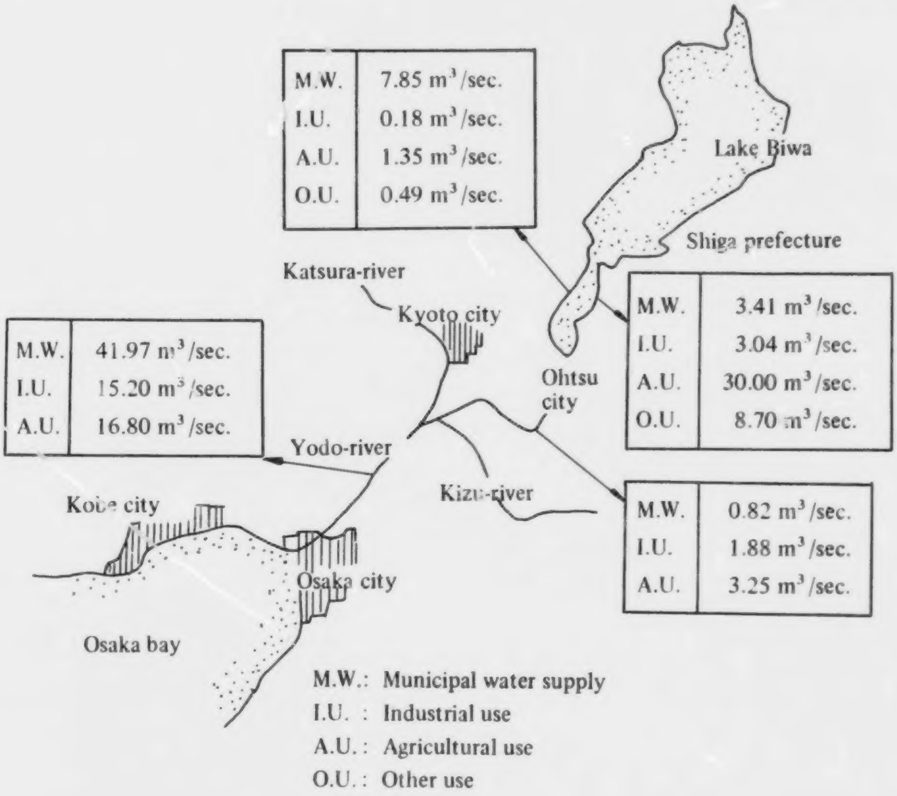
Table 9.1 Geographical and Other Statistics at Lake Biwa

Location	:	Shiga Prefecture
Normal water level	:	85.614 m above Osaka Bay datum, Osaka pile low water level
Total Area	:	680 km^2
Main basin	:	610 km^2 (Northern Part)
Sub basin	:	70 km^2 (Southern Part)
Total basin area	:	3848 km^2
Total coast line	:	240 km
Maximum depth	:	103.58 m
Average depth	:	41.20 m
Volume of water	:	27.5×10^9 m^3
Retention time	:	5.2 year

Lake Biwa is very important water resource for navigation, irrigation, domestic water supply and industrial purposes.

Fig. 9.1 Water Usage from Lake Biwa and Yodo-River

(based on water rights) June 1972



M.W. 7.85 m³/sec.
I.U. 0.18 m³/sec.
A.U. 1.35 m³/sec.
O.U. 0.49 m³/sec.
Lake Biwa
Shiga prefecture
Katsura-river
Kyoto city
M.W. 3.41 m³/sec.
I.U. 3.04 m³/sec.
A.U. 30.00 m³/sec.
O.U. 8.70 m³/sec.
M.W. 41.97 m³/sec.
I.U. 15.20 m³/sec.
A.U. 16.80 m³/sec.
Ohtsu city
Yodo-river
Kizu-river
Kobe city
Osaka city
M.W. 0.82 m³/sec.
I.U. 1.88 m³/sec.
A.U. 3.25 m³/sec.
Osaka bay
M.W.: Municipal water supply
I.U. : Industrial use
A.U. : Agricultural use
O.U. : Other use

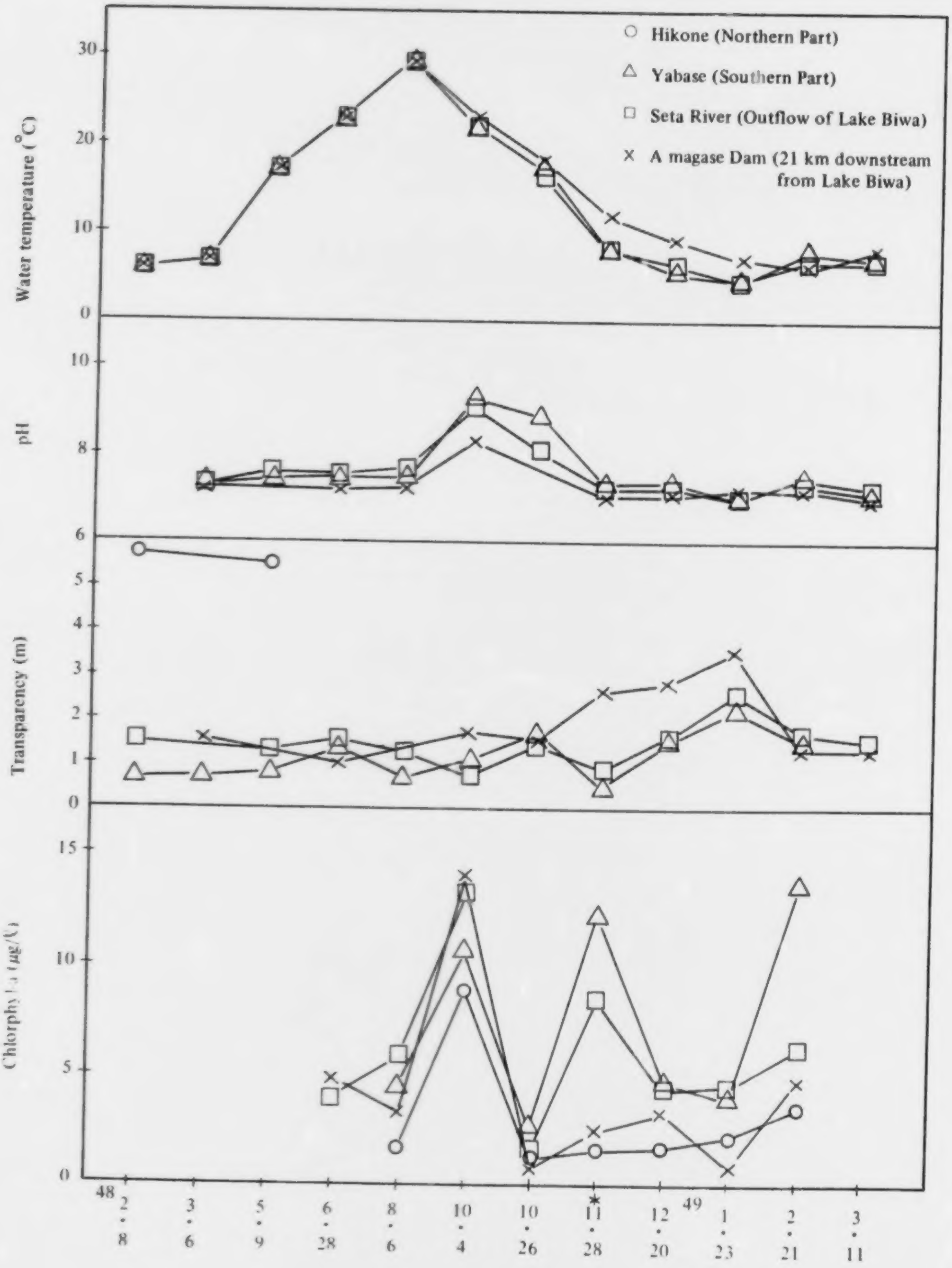


Fig. 9.2 Monthly Change in Water Quality–1

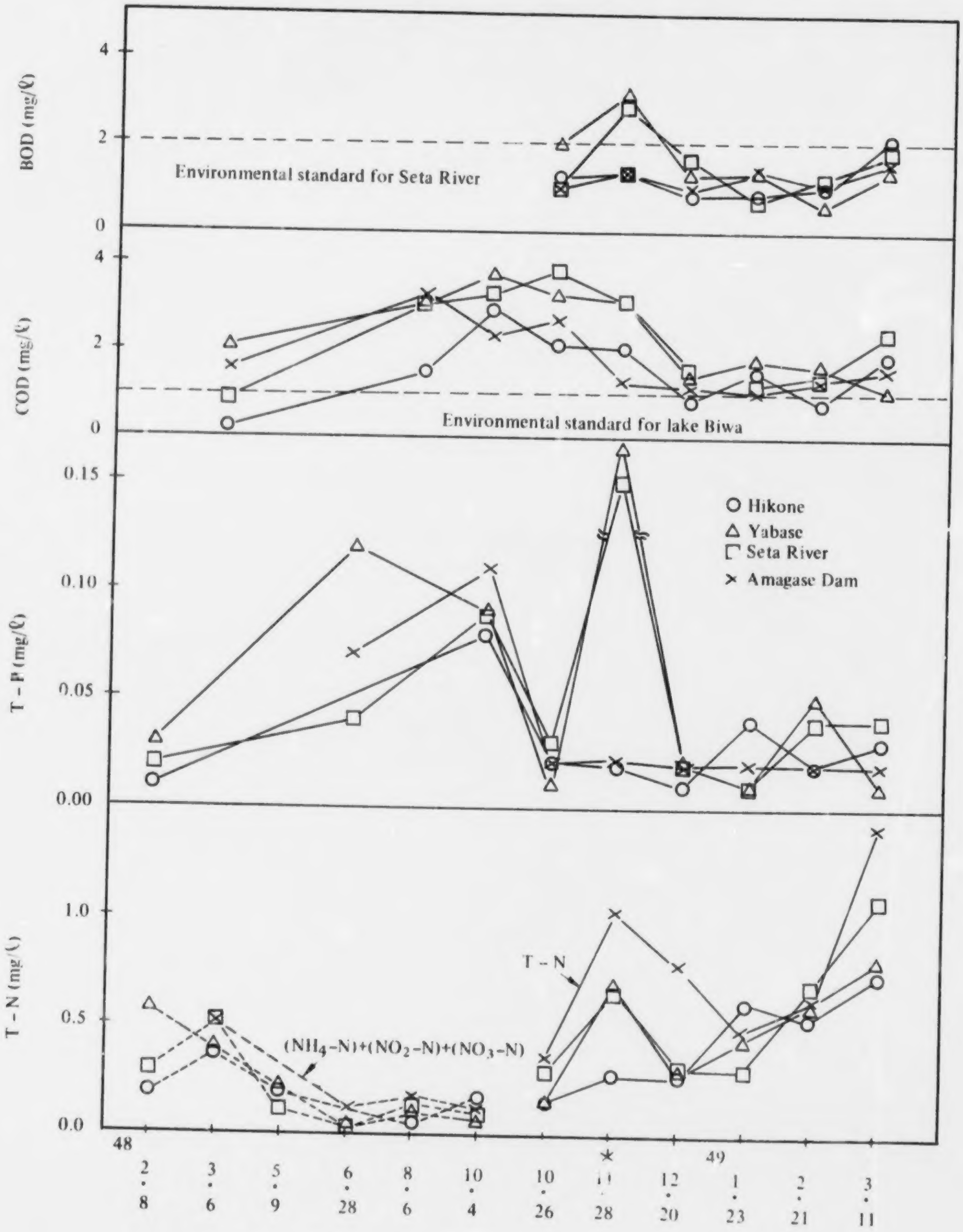


Fig. 9.2 Monthly Change in Water Quality – 2

9.2 PRESENT SITUATION OF WATER QUALITY AT LAKE BIWA

Fig. 9-2 shows the results of a water quality survey at the Biwa and at its outflow, the Seta River. The survey was conducted from 1973 to 1974.

The water temperature is found to stand at as low as 5 to 6 degrees Centigrade between January and March, then start rising gradually some time in May and reach almost 30 degrees Centigrade in August. Then, it starts down gradually. There appears no remarkable distinction in the temperature around the lake. pH value is at approximately 7.0 to 7.5 at every point, but from summer to autumn it goes slightly up to 8 to 9. The rise of pH-value has something to do with the growth of algae.

It seems probable that pH-value rose after CO_2 underwater had been reduced due to abundant growth of algae. The transparency of Biwa Lake water is low in its southern part. It is approximately 2.0 meters or less all through the year. Although it improves a little from November to January, the value does not exceed more than 2.5 to 3.0 meters. Considering the fact that the transparency exceeds 40 meters at Lake Mashu in Hokkaido Lake Baikal, in U.S.S.R., both of which are said to maintain the highest transparency in the world, it can be concluded that the Biwa has been polluted to a great extent. Even in the northern part where water is said to be relatively clean, the transparency at Lake Biwa stands at about 5 meters. This leads us to consider that northern Biwa is transforming from an oligotophic lake to a mesotrophic lake. Chlorophyl is a pigment which plays a main role in the photosynthetic reaction of plants, and is classifiable into four kinds; namely, a, b, c and d. Among these four, chlorophyl-a is contained in every kind of algae and accordingly is used to indicate the standing crop of algae in a water basin. The content of chlorophyl–a at Lake Biwa, though slightly variable according to the data, was 1 to 3 micrograms (μg) per liter in the northern part and 5 to 13 micrograms per liter in the southern part. The content of chlorophyl at lakes and marshes is reported by Ichimura and his associates as follows:

	(concentration of Chlorophyl-a)
oligotrophic lake	0.1 to 0.8 μg/l
mesotrophic lake	1 to 5 μg/l
eutrophic lake	10 to 60 μg/l

Chlorophyle at Lake Suwa, the most eutrophic lake in Japan, shows 30 to 60 micrograms per liter between June and September. With this in view, it is judged that the northern part of Lake Biwa has already turned into a mesotrophic lake and the southern part into a eutrophic lake respectively. There was a point where the highest BOD (Biochemical Oxygen Demand) stood at 3 milligrams per liter, but in general each point showed such a relatively low value as 1 to 2 milligrams per liter. The environmental water quality standard for Lake Biwa has been decided to be less than 1 milligrams per liter in terms of COD ($KMnO_4$-COD), but at any points the COD value in the lake water exceeds the standard and especially from August through November it goes as high as 3 to 4 milligrams per liter. As compared to the result of a survey in 1963 in which the annual mean COD indicated 1.12 milligrams per liter in the southern part of Lake Biwa, it is positively presumable that the pol-

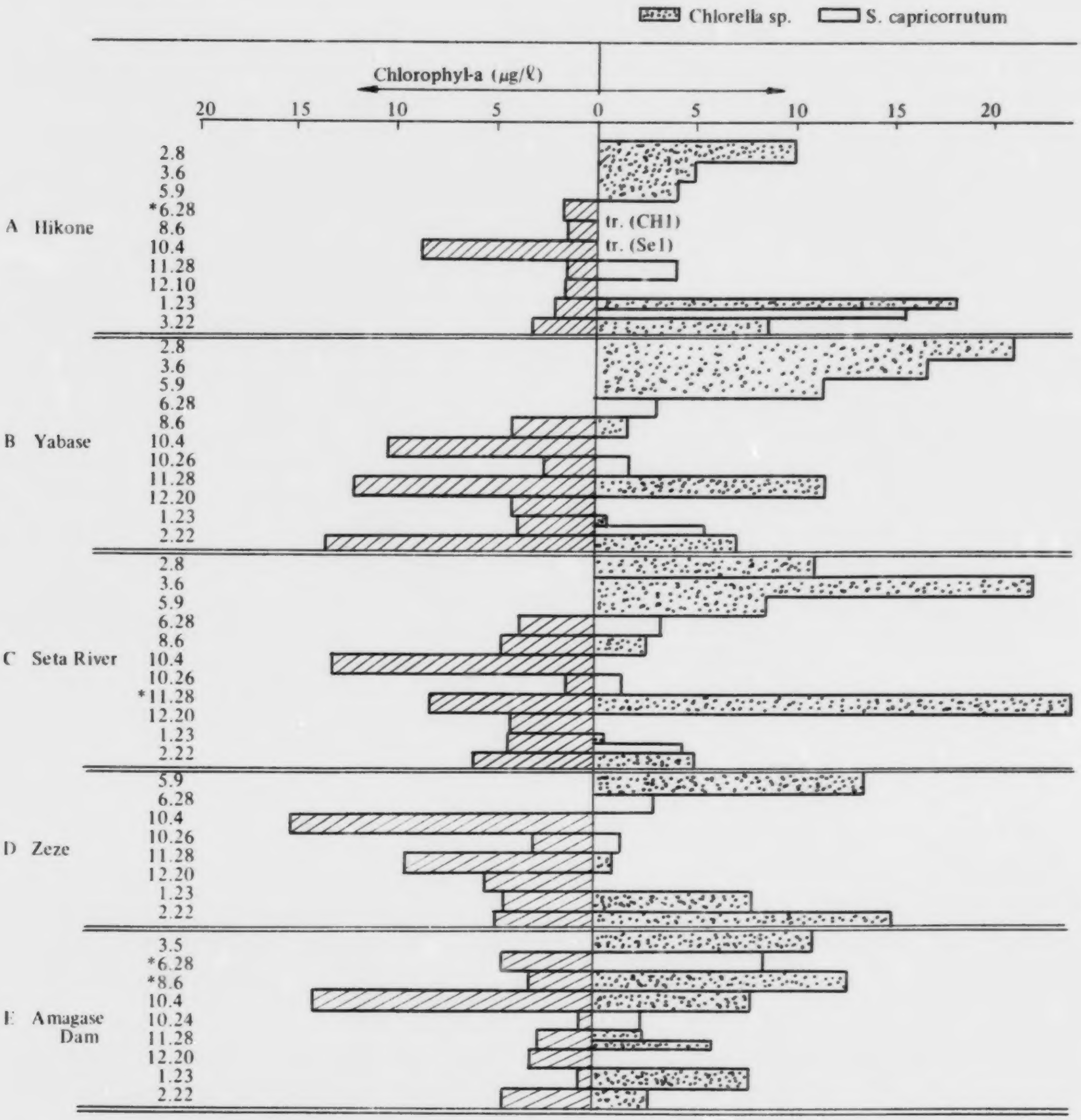


Fig. 9.3 Comparison of The Existing Volume of Algae and AGP

A: Hikone Northern Part
B: Yabase......... Southern Part
C: Seta River.... Outflow
D: Zeze............ Southern Part
E: Amagase Dam..... 21 km downstream

lution is in progress at Lake Biwa. The reason why COD stays low in winter and high in summer and autumn seems to be because there appears an increase of COD owing to the growth of algae. T-P is said to have interrelations, to a certain extent, with the degree of eutrophication, since the valute of T-P includes phosphorus in the micro-organism grown in the same water basin. And this is why T-P shows generally a high value from summer through autumn when COD also stays high. In general, NH_4-N is the indication that the water is polluted with aninal excreta or agricultural water. However, since NH_4-N is also oxidizable into NO_2-N and NO_3-N by the action of bacteria, total inorganic nitrogen or T-N should be ingnored. Total inorgani nitrogen stood at approximately 0.2 milligram per liter or less in the northern part of Lake Biwa all year round. whereas in the southern part it showed as low as 0.1 milligram per liter in autumn on the contrary to such a high value as 0.6 milligram per liter be between February and June. This seems to have resulted from the reduction caused by the growth of algae, as total inorganic nitrogen is normally consumed by algae. While chlorophyl shows the existing volume of algae, i.e., the volume in existence of algae appeared in the current lake water the AGP (Algal Growth Potential) indicates a potential of the algal growth. The sum of the two may surely correspond to the "total force of eutrophication". Fig. 9.3 shows relations between the present algae concentration and AGP. AGP at Hikone (in the northern part) during the wintertime stands at almost 10 milligrams per liter or less. This means that for the prevention of eutrophication, AGP should be kept at less than 10 milligrams per liter during the wintertime. To stop eutrophication in the southern part it seems necessary to keep AGP down to 20 milligrams per liter or less. Further, at Lake Suwa which is said to have been most eutrophicated in Japan, the water quality in terms of COD is 5 to 8 milligrams per liter, 0.2 to 0.3 milligram per liter in in T–P, about 0.20 milligram per liter in PO_4-P, 0.5 to 2.0 milligrams per liter in NH_4-N and 11 to 38 milligrams per liter in AGP (Chl.), respectively. The southern part of Lake Biwa seems to have already been polluted as badly as Lake Suwa.

As has been mentioned above, the pollution at Lake Biwa is severe. The urgent countermeasures are in need now so as to prevent the further pollution and eutrophication. Accordingly, the Japan Sewage Works Agency, having been entrusted by the Ministry of Construction and the Shiga Prefectural Government, started surveys and research works to prevent the pollution at Lake Biwa, especially its eutrophication.

9.3 EUTROPHICATION CONTROL AND EFFLUENT QUALITY STANDARD

Sewage Works Act states that publicly owned treatment work provide treatment with effluent limitations. The effluent standard for secondary treatments such as activated sludge process is shown in Table 9.2

Fig. 9.4 **Effects of Adding of Secondary Effluent (Activated Sludge) or Tertiary Effluent (Alum Coagulation)**

Algae Inoculated: Chlorella SP.
Treatment Effluent Adding: 5 % (volumetric)

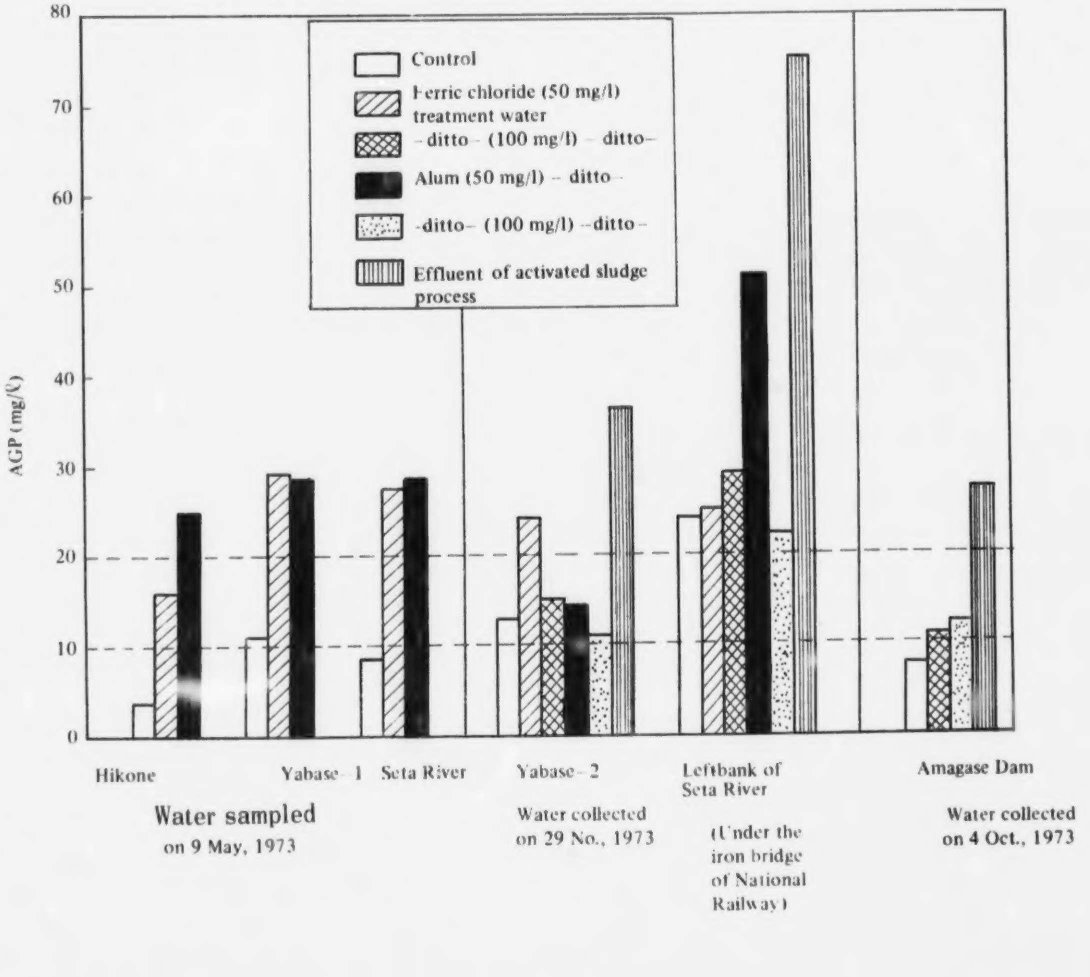


Table 9.2 Sewage Treatment Plant Effluent Quality Standard

Category	Standard Value
H	5.8 ~ 8.6
BOD_5	less than 20 mg/l
SS	less than 70 mg/l
counting Coli group	less than 3,000 N/ml

BOD and SS can be removed efficiently in the activated sludge process. The process can produce the effluent that fully satisfys the effluent quality standard established by the Sewage Works Act if only due attention is paid to the operation and maintenance. However, the effluent standard is now out of date. Recently the environmental water quality standard is set up in the whole Biwa Lake.

The actual allowable effluent quality is a different story from the traditional effluent standard. The allowable effluent quality has to be determined as a standard necessary to satisfy the environmental water quality standards established in full consideration of various factors and their mutual relation, etc. The environmental standards at Lake Biwa and its outflow, the Seta River are as shown in Table 9.3

Table 9–3 Environmental Water Quality Standard at Lake Biwa and Seta River

Lake Biwa	pH	COD mg/l	SS mg/l	DO mg/l	Coli. Counts MPN/100 ml	Target Date
Northern Lake	6.5–8.5	less than 1.0	less than 1.0	more than 7.5	less than 50	*
Southern Lake	6.5–8.5	less than 1.0	less than 1.0	more than 7.5	less than 50	1977
Seta River (Outflow)	6.5–8.5	less than 1.0	less than 25.0	more than 7.5	less than 1000	*

* Immediately satisfied after issued standard in 1972

As mentioned in the preceding chapter, the value of COD at Lake Biwa is much greater than that of environmental standard, 1 milligram per liter, in both the northern and southern parts. Especially during the summer period it stands at 2 to 4 milligrams per liter. In order to achieve this environmental standard at the earliest possible time, it is in urgent need to reduce the pollution load. On the basis of the data about the self-purification, the dilution, and the diffusion the actual effluent quality standard should be set up. Then required waste water treatment process will be designed. The environmental standard and the effluent quality standard on phosphorus and nitrogen are scheduled to be established in the near future although the present standard says nothing oth them.

Thus, to prevent the eutrophication, the nutrients should be removed in nearest future. It is said that ferric salts, vitamins and carbonate also have relations with the eutrophication beside from nitrogen and phosphorus, but as of now the requisite information is insufficient. In general, nitrogen and phosphorus are considered to be the limiting factors of the eutrophication.

The result of a study on the limiting nutrients at Lake Biwa by means of the AGP Method is shown in Table 9.4 According to this result, since even at the same point of Lake Biwa nitrogen plays a role of the limiting factor in one season and then phosphorus takes its place in another season, it is considered that there exists a variation depending on the geographical conditions, the kinds of pollution and the meteorological conditions, etc. in the lake.

Table 9.4 Restrictive Factors of the Growth of Algae

		Phosphorus	Nitrogen
Hikone (Northern Part)	August 1973 October	●	○
	October	●	○
Yabase (Southern Part)	June Augus	●	○
	August	○	●
Seta River (Outflow of Lake Biwa)	June	●	●
	August	○	○
	January 1974	○	●
Zeze (Southern Part)	October 1973	●	○

● Stimulate the growth of algae
○ Does not stimulate the growth of algae

Accordingly, it will be known that the removal of both nitrogen and phosphorus is desirable for the complete control of the eutrophication. However, due to the existence of bacteria and algae (blue-green algae) which fix it from the air, it is difficult to control the nitrogen efficiently. Hence, studies were herein conducted before everything on the removal of phosphorus.

Now, there arises a question; to what extent the concentration of phosphorus in the effluent water from a sewage treatment plant should be lowered? This question can be answered rather easily if the AGP is applied.

By changing alum dosage, effluents which contain several concentrations of phosphorus are prepared. Then the effluents are dilluted with the least eutrophicated lake water at a certain ratio, say 20 times. When AGP is related with the phosphorus concentration in the prepared effluent, allowable maximum phosphorus concentration would be determined. Fig. 9.4 shows the results of the examination.

As is shown in the figure, when an amount of either alum or ferric chloride is dosed to reach its concentration of 100 ppm, AGP in the mixture is always kept less than 20 mg/l. For this circumstance, phosphorus concentration in the prepared secondary effluent is 0.03 mg/l. Judging from this result, it is considered that the eutrophication in the lake water may hardly be promoted if PO_4-P as the tertiary treatment water is set at lower than 0.03 to 0.02 milligram per liter.

9.4 REMOVAL OF PHOSPHORUS

9.4.1 BACK GROUND

For the purpose of phosphorus removal from urban sanitary sewage, technically and economically feasible processes are the following

a. Luxury biological phosphorus uptake process
b. Coagulant dosing process
c. Lime precipitation process
d. Alum precipitation process

It was reported that if the luxury uptake process was successfully operated, over 80 percent of the total phosphorus could be removed. However it is not easy or not always possible to keep the process at the optimum condition. Sometimes the process may be of success in the luxury uptake but sometimes not.

By means of dosing coagulant to an aeration tank, the phosphorus removal through the secondary treatment process improves considerably. At first we believed that this was an economical way to grade up the present facility. We tried to examine performance of this process on a pilot plant. When three times aluminum in mol ratio to phosphorus were dosed to the aeration tank, about 60~80 percent of the phosphorus existed in inflow sewage were precipitated. If this amount of alum was added, its concentration in the mixed liquor would be about 150 mg/l. Even though such the high dosage as 150 mg/l, the secondary effluent still contained 0.2 ~ 0.5 mg/l of phosphorus. This concentration was not satisfactory for the Biwa Lake water. Higher phosphorus removal required more alum dosage. A new problem rised; that was retardation of biological activity. Too much alum in the biological process resulted in poor BOD removal. Thus, we quitted this process and came to the decision that an independent advanced plant should be placed to cope with the phosphorus problem.

The most popular and effective coagulant for this purpose is lime. Sludge produced from the lime precipitation process can be easily dewatered by ordinal devices. This is one of the important advantage in selecting any unit processes. Disadvantages being associated with this process are difficulty in handling lime, and scaling in pipes.

The alternative is the alum precipitation process. For the phosphorus removal alone, alum is as effective as lime. Alum is easy to handle and is not scaling. However on the other hands, alum sludge is hard to dewater.

Finally we came to the conclusion that the alum process was selected for the case of Lake Biwa's tertiary purpose. It is not because we like the advantages of the alum process more, but because we like the disadvantages of the lime process less. There was no news of success in dewatering sole alum sludge. Development of sludge dewatering engineering and establishment of design parameters for the alum process were the next step to approach the final goal. This was a story untilthis pilot plant was placed in Otsu sewage plant.

9.4.2 PILOT PLANT FOR PHOSPHORUS REMOVAL

A flow sheet of the pilot plant is shown in Fig. 9.5 The hydraulic capacity is 500 cubic meters per day. In this flow rate, design detention time is 15, 30, and 150

Fig. 9.5 Flow Diagram of Coagulation Process

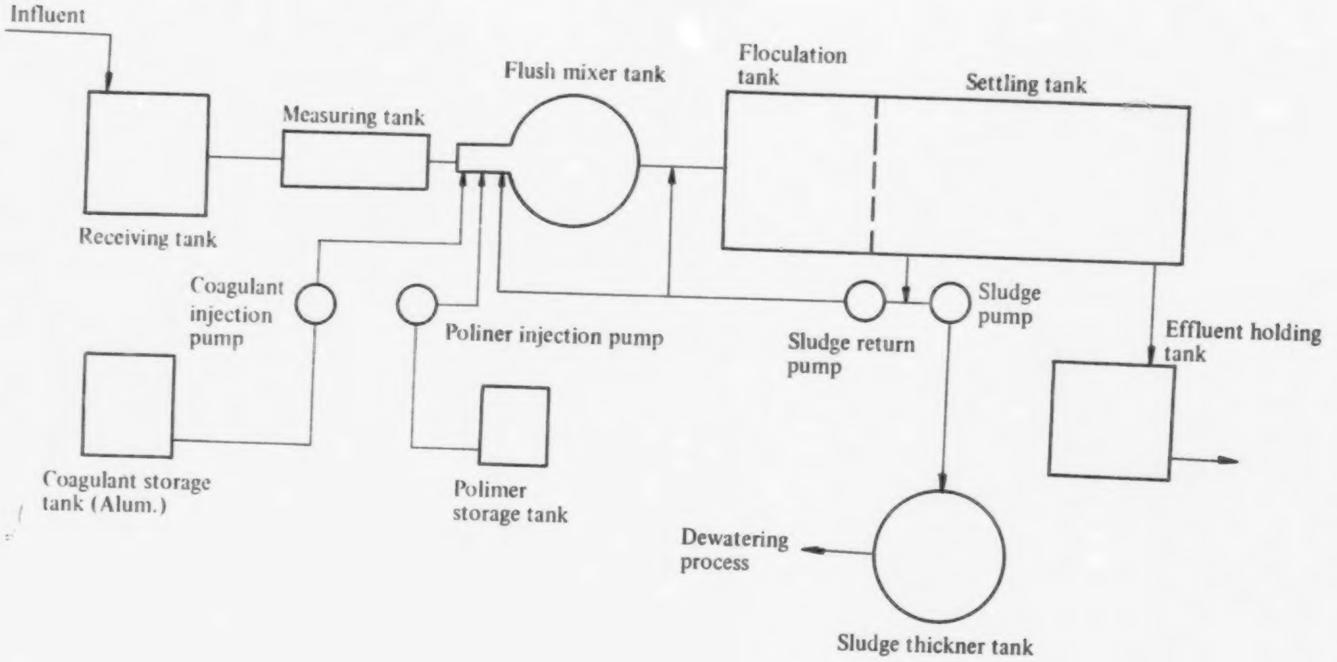
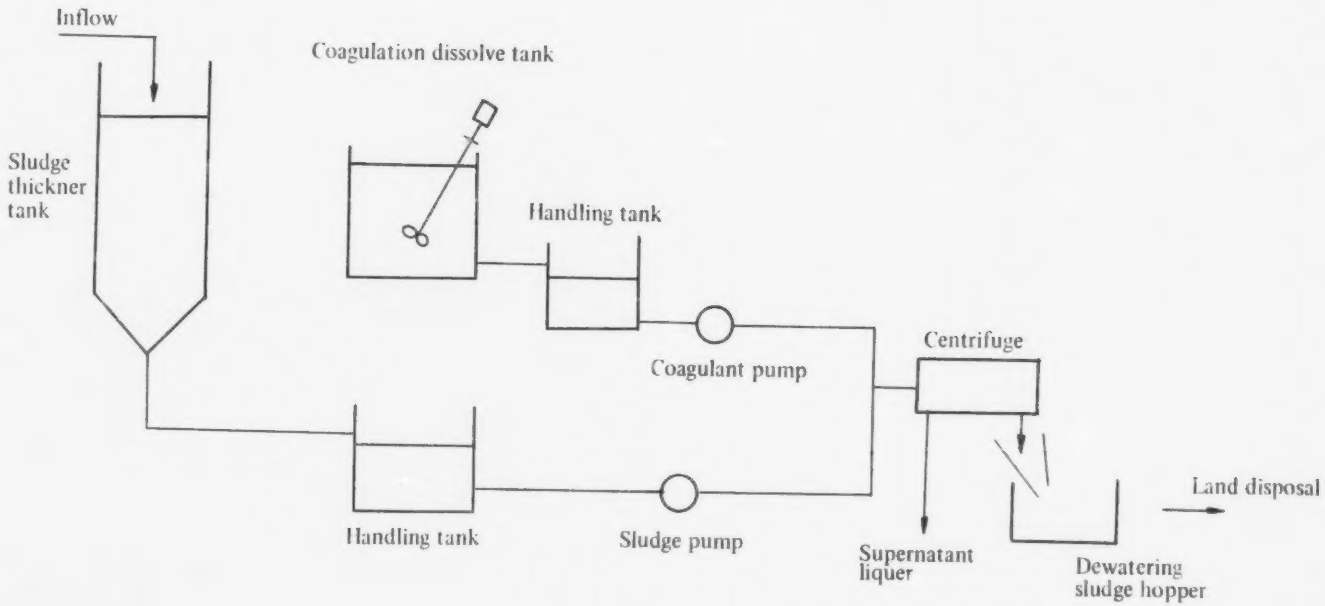

Influent
Receiving tank
Measuring tank
Flush mixer tank
Floculation tank
Settling tank
Coagulant injection pump
Poliner injection pump
Sludge return pump
Sludge pump
Effluent holding tank
Coagulant storage tank (Alum.)
Polimer storage tank
Dewatering process
Sludge thickner tank

Fig. 9.6 Flow Diagram of Sludge Handling Process


Inflow
Sludge thickner tank
Coagulation dissolve tank
Handling tank
Coagulant pump
Handling tank
Sludge pump
Centrifuge
Supernatant liquer
Dewatering sludge hopper
Land disposal

minutes for the rapid mixing tank, the flocculation tank, and precipitation tank respectively.

Into the center of the rapid mixing tank, liquid alum and polymer are fed at a certain rate through chemical feeding pumps. A small amount of the polymer is helpful with alum flocculation.

In order to save the alum, some portions of the precipitated alum sludge are to be returned from the precipitation tank to either the rapid mixing tank or the flocculation tank.

In future, automatic control system will be introduced so that the alum can be dosed proportionally to phosphorus concentration. Thus, we will be able to examine several aspects on the automatic control practice.

9.4.3 PHOSPHORUS REMOVAL

In general, a large plant requires much more alum that of theoretical requirement due to improper design. Our main concern is to find the optimum process conditions under which phosphorus can effectively be removed by a small amount of alum dosage. Up to the present, it is found that when alum is dosed as much as 13.6 mg/l (or mol ratio to phosphorus 5 to 1) about 90 percents phosphorus are removed.

The pilot study is now under ways for the purpose of finding:

(1) A relation between sludge producing rate and alum dosage
(2) a relation between SS removal rate and alum dosage
(3) effect of return sludge on phosphorus removal and on alum saving
(4) optimum rapid mixing and slow mixing
(5) polymer dosing rate and dosing point.

9.5 SLUDGE HANDLING

9.5.1 ALUM SLUDGE

An increase in alum dosage results in an increase in phosphorus removal. We can cope with any requirement for high phosphorus removal. Removing the phosphorus in a rather easy job. One of the big problems included in the alum precipitation is that a large amount of sludge is produced. The amount is directly proportional to the alum dosage. Through the course of the present pilot alum process, in every 500 cubic meters influent, 3.4 cubic meters sludge are produced. Therefore when 1 million cubic meters raw sewage is treated in the actual plant, daily sludge production rate will be 7000 cubic meters. A special attention should be paid to this large number. A success in phosphorus removal from Otsu's effluent will be entirely dependent on the sludge handling capability.

The kinetics for the alum precipitation process is not yet completely clear. It seems probable that two reactions may be concerned with this process; (1) alum combines directly phosphorus and forms flocs of aluminum phosphate, and (2) phosphorus is absorbed at the surface of aluminum hydroxide which is formed through a reaction between the alum and alkalinity.

Phosphorus (total–p) concentration in the secondary effluent from the present Otsu sewage plant is as low as 1 mg per liter. Suspended solid concentration ranges from 3 to 5 mg per liter. Volatile solid contents are about 35 percents of the total suspended solids. This means that large portion of the precipitated sludge is aluminum compound.

9.5.2 THICKENING

Underflow concentration in the precipitation tank is no greater than 5000 mg per liter. The sludge is very hard to thicken; under the quiescent condition, by gravity, during 24 hours, an increase in solid concentration is about 30 or 40 percentration never exceeds 10000 mg per liter by the gravity thickening method.

9.5.3 DEWATERING

The solid particles in the sludge are very fine and they stay as soll or gell in it. Leaf tests do not show any capability for vacuum filters to dewater the sludge. It is observed that the fine particle included in the alum sludge pass through the filter cloth when a Buchner funnel test is performed. Magnitude of the drainability or filterbility can not be determined in the laboratory Buchner funnel test.

We give up examining possibilities of success in dewatering by using the vacuum filter. Our interests are shifted to centrifuge dewatering. It may be the only device which has possibility to yield satisfactorily moistured sludge cake from alum precipitant. Meanwhile, our laboratory research shows that if the polymer is properly selected and properly dosed, a certain centrifuge will do the job. Polymer is a very important factor to affect centrifuge performance. Our staffs moves up from the laboratory to the field where the pilot plant is located. Fig- 9.6 is a flow sheet of the plant. The dewatering study continues with this plant now.

The findings are summerized in Table 9.5. The tests from run # 1 to # 5 are for selecting most suitable polymer and those from # 6 to # 9 are for optimum operational conditions of the centrifuge.

The design specification of the pilot centrifuge are as follows;

Maximum Power	3200 G
Maximum Revolution Speed	6000 RPM
Motor Output	3.75 KW
Capacity	500 liter/hr.

The followings are concluded from the pilot study:

(1) More than 96 percents of the suspended solid are recovered and moisture concentration in the dewatered sludge cake is 90% at average, (85% at minimum). The dewatered sludge can sustain own shape by itself on a flat plate. This can be handled by hands or a shovel easily.

(2) An increase in the polymer dosage rate does not improve the moisture concentration. On the basis of dry solids, 0.5 percents are the optimum dosing rate.

(3) If the sludge feeding rate is increased, the moisture concentration in the dewatered sludge is kept fairly constant value, 90%, and the SS recovery rate is decreased. The same is observed when difference of revolution speeds between the bowl and the conveyer is increased.

(4) Whether cathionic or anionic is not an important factor. The effectiveness of the polymer is not dependent of its ion charge.

(5) Solid concentration of the fed sludge is a main factor on the centrifuge performance. The thicker is the fed sludge, the less is the moisture contents of the dewatered sludge.

(6) With lower bowel dam height in the centrifuge than with higher, less moistured sludge are yielded. The pilot centrifuge is designed to adjust the dam height at 4 levels. With the lowest level of the dam, least moistured sludge are produced. It is around 85 percents.

The value of 85 percents moisture concentrations in the dewatered sludge is not perfectly satisfactory. Our efforts will be concentrated at reducing the number to about 80 percents. Selection of proper polymer is a way to approach the purpose. Another way is to stimulate manufacturers to provide a proper centrifuge.

One of the important thing that we should know is how Acrylamid, main component of the polymer, affects the environment. When supernatant is returned to the secondary process, at present moment, we are not sure what happens with the secondary effluent. We are going to examine bio-degrability of the polymer returned to the aeration tank.

Commonly it is reported that alum recovery is not economically feasible. However, from the stand point of natural resource saving, our interests are still kept on developing the alum recovery engineering. Sulfic acid process could be in higher potential.

Table 9.5 Summary of Sludge Dewatering Performance

Test No.	Spec. of Machine			Influent			Filtrate	SS		Coagulant			
	Revolution (r.p.m)	Dam No.	Revolution Difference	Density (%)	Sludge loading	Solid loading	Density (%)	Recovery ratio	Moisture (%)	Density (%)	Volume (l/hr)	FA/SS (%)	Sort
1	5000	4	(r.r.m) 6	0.6260	(l/hr) 400	(kg/hr) 2,505	0.0245	96.09	90.07	0.05	30	0.600	A
2	5000	4	6	0.6230	400	2,490	0.0220	96.47	89.95	0.05	55	1.105	B
3	5000	4	6	0.4800	400	1,921	0.0136	97.17	88.79	0.05	34	0.916	C
4	5000	4	6	0.6970	400	2,788	0.0146	97.91	89.21	0.05	31	0.556	D
5	5000	4	6	0.6770	400	2,732	0.0090	98.67	89.36	0.05	31	0.568	E
6	5000	4	10	0.6200	400	2,480	0.0450	92.74	90.79	0.05	30	0.6048	A
7	5000	3	6	0.9190	500	4,595	0.0113	98.77	88.14	0.10	17	0.3680	B
8	5000	3	6	0.9740	500	4,870	0.0084	99.91	86.82	0.05	80	1.643	C
9	5000	3	15	0.8810	700	6,167	0.0300	96.59	88.54	0.10	56	0.908	B

A: NO–75° (nonion)
B: A–101 (anion)
C: N–800 (nonion)
D: SS–200 (nonion)
E: C–011K (cation)

9.6 FURTHER RESEARCH NEEDS

9.6.1 LOADING OF PHOSPHORUS AND NITROGEN ON LAKE BIWA

Kinki Branch Office of Ministry of Construction conducted intensive study on nutrient loading on Lake Biwa in the last year as shown in Table 9.6 and 9.7.

Table 9.6 Total Phosphorus Loading*

	1970		1980**	
	Load (kg/d)	Share (%)	Load (kg/d)	Share (%)
Domestic Waste Water	174.0	51.9	247.5	44.6
Industrial Warte Water	21.6	6.4	125.6	22.6
Warte-water from eivestock Yard	32.8	9.8	74.3	13.4
Discharge from Pady Field	40.3	12.0	41.9	7.5
Discharge from Frest etc.	66.3	19.8	66.3	11.9
Total	335	100	555.6	100

Table 9.7 Total Nitrogen Loading*

	1970		1980**	
	Load (kg/d)	Share (%)	Load (kg/d)	Share (%)
Domestic Waste-water	1960.5	33.1	1854.5	28.4
Industrial Waste-water	801.5	17.0	1974.9	30.2
Waste-Water livestock yard	111.5	2.4	244.6	3.7
Paddy field	974.6	20.7	1181.8	18.1
Frest	1264.9	26.8	1264.9	19.4
Total	4713	100	6520.7	100

*(1) Table does not include load from well water source because of uncertainty of measurements.

(2) This figure are projected before treatment.

** This is estimated based on economic development plan of Shiga Pufectural Government.

The water quality of lake Biwa at present are mentioned before, it will be expected that nutrients loading exceed a certain amount very much such as critical or dangerous level as proposed Vollenweider. The rate of effluent to total discharge of Lake Biwa to downstream is around 10%, so it is very important to remove the phosphorus to protect lake water.
Unfortunately available data are very scarce on water quality, flow pattern, diffusion characteristies and budget of nutrients in the lake, much research works are needed in this fields.

9.6.2 FURTHER PILOT PLANT STUDY

Pilot plant study for Lake Biwa is 4 Years project, then it will be completed in 1978. First of all, removal of phosphorus by alum precipitation are examined, and dewatering of alum sludge are found to be possible by centrifuge. All possible alternatives to remove nutrients from wastewater should be projected to examine including cost effectiveness study in this project.

Reference.

1) J.S.W.A. Survey on the Pollution in Lake Biwa in 1972, 1973.
2) An Approach to a Relative Trophic Index System for Classifying Lakes and Reservoirs, Working Paper No.24. Pacific North West Environmental Research Laboratory U.S. EPA.

Fourth US/JAPAN Conference
on
Sewage Treatment Technolgy
Paper Annex to No. 4

CASE STUDIES ON PCB POLLUTION

October 29, 1975
Washington, D.C.

Ministry of Construction
Japanese Government

CASE STUDIES ON PCB POLLUTION (CONTINUED)

2.2 CASE STUDIES ON PCB POLLUTION (continued)

Environmental pollution of PCBs was first recognised in Europe followed by the United States then Japan. Results of field investigations demonstrated that PCB pollution in organisms inhabiting the natural environment was at a severe level. In Japan, analytical results indicating concentrations of residues in wild animals were first reported in 1971. In January of 1972, a standard method for the analysis of PCBs was established by the reseach group of the Ministry of Health and Welfare, whereby the uniform analysis of PCB residues in wild animals was realised.

A national survey concerning environmental pollution of PCBs was conducted from May to December, 1972. During this period, the water quality of 1,084 locations and bottom sediments sampled from 1,445 places was inspected. Canvassing the agricultural industry, PCB analysis were carried out for samples of soil and rice taken from 88 locations throughout the country, while similar analysis were conducted for 559 samples of fisheries products taken from 110 aquqtic regions. Summaries of these results are shown in Table 4, 5 and 6. From 1973 to March, 1974, 1,281 samples were collected from 282 aquatic regions for water quality analysis (208 rivers; 28 harbours; 46 ocean regions), while 1,789 samples of bottom sediments were collected from 354 aquatic regions (258 rivers; 38 harbours; 58 ocean regions) as well as 3,369 samples representing aquatic biota.

In Japan, the accumulation of PCBs in the human body can be surmised and evaluated on the basis of analytical results of residues detected in human milk. Such a survey was taken throughout the country during the three year period from 1972 to 1974, and the results are summarised in Table 7. A summary of regional differences based upon the results of the 1974 survey is presented in Table 8, and indicates that the PCB content of human milk tended to be highest in those regions described primarily as fishing villages and lowest in farm inhabitants.

Table 4 Results of the Survey on PCB Pollution (Water Quality and Sediments)

PCB content (ppm)	Water Quality			Sediments							Grand Total
	0.01 or less	0.01 or more	Sub-total	1 or less	1.1 ~ 10	11 ~ 50	51 ~ 99	100 ~ 499	500 or more	Sub-total	
Industries using PCB	78	13	91	0	19	10	5	11	9	54	145
Sewage treatment plants	125	0	125	-	-	-	-	-	-	-	125
Waters nearby factories	76	4	80	26	24	15	7	1	4	77	157
Public Waters	785	3	788	1,216	73	20	2	3	0	1,314	2,102
Total	1,064	20	1,084	1,242	116	45	14	15	13	1,445	2,529

Table 5 Results of the Survey on PCB Pollution (Soil and Agricultural Products)

PCB Concentration	Number of Soil Samples	Number of Unhulled Rice Samples
less than 0.01 ppm	35 (40%)	26 (79%)
0.01 0.10 ppm	21 (24%)	5 (15%)
0.11 1.0 ppm	19 (21%)	1 (3%)
1.1 10.0 ppm	6 (7%)	1 (3%)
10.1 100.0 ppm	3 (3%)	
more than 100.1 ppm	4 (5%)	
Total	88 (100%)	

Table 6 Results of the Survey on PCB Pollution (Fisheries Products) (1972)

PCB Concentration (ppm)	0.09 or less	0.1 ~ 0.4	0.5 ~ 0.9	1-2	3	more than 3	Total
Sea Water	(34) 155	(38) 175	(12) 53	(11) 48	(2) 11	(3) 14	(100) 456
Fresh Water	(15) 21	(44) 63	(23) 33	(13) 19	(1) 2	(4) 5	(100) 143
Total	(30) 176	(40) 238	(14) 86	(11) 67	(2) 13	(3) 19	(100) 599

Table 8 Frequency Distribution of PCB Concentration in Human Milk in Urban, Agricultural and Industrial Regions(1972)

Concentration (ppm)	Total (Cumulative %)	Urban Residential Areas	Farm Villages	Fishing Villages	Industrial Vicinities
0.009	67(12.1)	22	34	8	3
0.01	86(27.5)	37	31	13	5
0.02	148(54.1)	74	49	17	8
0.03	90(70.3)	41	23	22	4
0.04	77(84.2)	36	23	15	3
0.05	33(90.1)	15	6	7	5
0.06	14(92.6)	6	5	3	-
0.07	25(97.1)	8	2	11	4
0.08	6(98.2)	3	1	2	-
0.09	4(98.9)	-	1	3	-
0.1	6(100.0)	2	1	3	-
0.2	-	-	-	-	-
Total	556	244	176	104	32
Mean	0.028	0.028	0.023	0.037	0.031
Median	0.023	0.024	0.020	0.031	0.025

Table 7 Comparison of the PCB Concentration in Human Milk from Year to Year in Urban, Agricultural and Industrial Regions

Concentration (ppm)	Urban Residential Areas			Farm Villages			Fishing Villages			Industrial Vicinities			Totals		
	'72	'73	'74	'72	'73	'74	'72	'73	'74	'72	'73	'74	'72	'73	'74
< 0.009	18	28	21	16	22	25	-	3	3	2	-	1	36	53	50
0.01	27	36	34	34	23	29	3	9	7	-	-	1	64	68	71
0.02	56	54	67	64	46	46	18	17	9	-	1	4	138	118	129
0.03	50	47	35	43	37	21	11	9	17	5	2	1	109	95	74
0.04	31	24	33	18	11	20	16	8	8	2	7	-	67	50	61
0.05	31	18	14	12	9	5	9	4	3	4	1	3	56	32	25
0.06	17	14	5	6	10	4	3	4	3	2	1	-	28	29	12
0.07	10	5	7	1	2	2	1	4	6	2	3	1	14	14	16
0.08	11	6	1	3	3	1	5	4	2	-	1	-	19	14	4
0.09	5	1	-	-	1	1	4	3	3	-	-	-	9	5	4
0.1	8	6	2	3	2	-	5	3	2	2	1	-	18	12	4
0.2	3	-	-	-	-	-	3	2	-	-	-	-	6	2	-
Total	267	239	219	200	166	154	78	70	63	19	17	11	564	492	447
Mean	.038	.030	.027	.026	.027	.023	.050	.044	.039	.047	.051	.031	.030	.032	.028
Median	.032	.025	.033	.023	.023	.020	.039	.032	.032	.048	.043	.024	.029	.026	.023

2.2.1 The Case in Lake Biwa

Lake Biwa is the largest lake in Japan and is centrally situated within the main island of Honshu. It covers an area of 695 square kilometres, has a maximum and average depth of 103.4 and 40.2 metres, respectively, and has a total water volume of 28.15 cubic kilometres. Many small cities such as Otsu, Hikone, Kusatsu and Nagahama with populations of less than 200,000 inhabitants are situated in the surrounding vicinity. The main industries of this region include synthetic fibres, textiles, electrical appliances, commerce and breeding of fisheries resources. Lake Biwa is also a water source which supplies the cities of Osaka, Kobe and Kyoto, and its shoreline is dotted by numerous recreation facilities.

(1) The region of the Seta River and Lake Biwa

The location of fifteen factories and plants which had previously used PCBs in some way of form are shown in Fig. 3. It is noteworthy that the majority were densely located in the southern region of the Lake Biwa. A survey of these factories revealed that thirteen had used KC 300 and 400 as a heat exchanger, while two had used KC 300 in the manufacture of electrical condensors. The total amount utilised exceeded 3,827 tons of which roughly 14 tons were suspected to have been discharged into the environment.

Table 9 gives estimated amounts of PCBs discharged in four vicinities located in or around the Seta River and Lake Biwa region. Properties of various commercial preparations of PCBs are given in Table 10 for reference. Table 11 shows concentrations of residues expressed as averages and ranges in water, sediments, fish, crustaceans, plankton and adhesive algae, while Table 12 presents residual values detected in some species of birds.

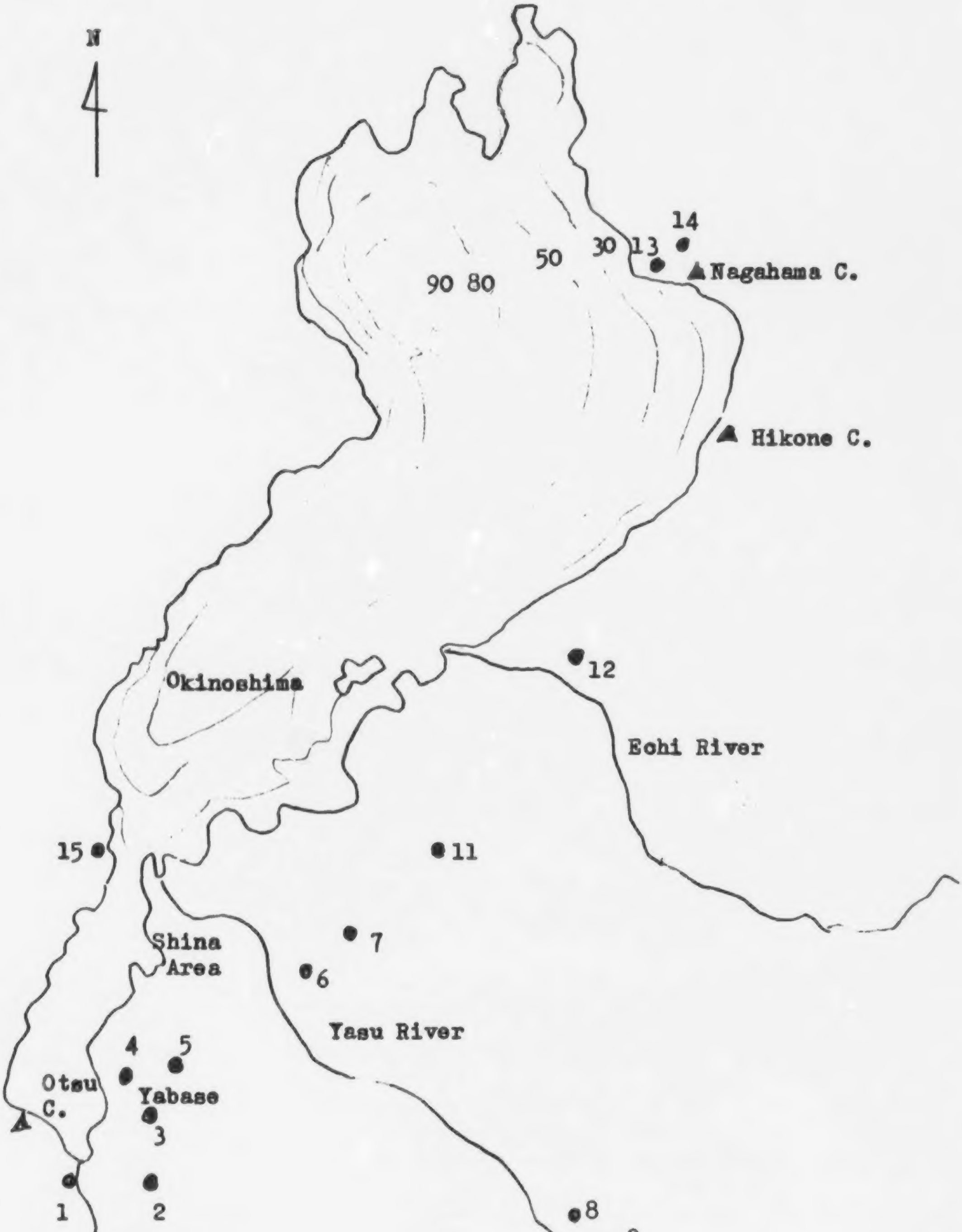

N
14
13
30
50
90
80
Nagahama C.
Hikone C.
12
Okinoshima
Echi River
15
11
Shina Area
7
6
Yasu River
4
5
Otsu C.
Yabase
3
1
2
Seta River
8
9
0
5
10
km
10

Fig. 3 The aquatic region of Lake Biwa and Seta River

Table 9 Scale of PCB Discharge into the Seta River and Lake Biwa Regions

Region	Seta River vicinity	Yabase, Niihama, Shina vicinity	Katada, Yoshikawa vicinity	Okinoshima vicinity
Period of Utilisation	1962 - 1972(Aug)	1954 - 1972(Oct)	1960(Nov) - 1972(Mar)	1965(Dec)-1972(Oct)
Amount Utilised (kg)	51,505	3,556,265	166,810	30,154
Amount Discharged into the Environment (kg)	42	7,320	37.22	3,316,164
Type of PCB Used	KC 300 KC 400 KC 500	KC 300 Santo-therm FR 1	KC 300, KC 400 KC 500 Santo-therm 1	KC 300 KC 400 Santo-therm 1,2
Purpose of Utilisation	Heat exchanger	Heat exchanger and Electrical condensers	Heat exchanger Electrical condensers and hardeners	Heat exchanger

Table 10 Physicochemical Properties of Kanechlors*

Kanechlor	Primary component (mixture of isomers)	Specific Gravity	Distillation Range (C @760 mm Hg)	Equivalent Monsanto Product
KC-200	Dichlorinated biphenyl	1.223 - 1.243	270 - 360	Aroclor 1232
KC-300	Trichlorinated biphenyl	1.310 - 1.322	325 - 360	Aroclor 1242
KC-400	Tetrachlorinated biphenyl	1.376 - 1.389	340 - 375	Aroclor 1248
KC-500	Pentachlorinated biphenyl	1.460 - 1.475	365 - 390	Aroclor 1254
KC-600	Hexachlorinated biphenyl	1.539 - 1.555	385 - 420	Aroclor 1260

*Product of the Kanegafuchi Chemical Industries

Table 11 Residues of PCBs in Water, Sediments, Fish, a Crustacean, Plankton and Adhesive Algae

Region	Seta River				Lake Biwa															
	N	min	max	ave	N	min	max	ave												
Water	6	ND																		
Sediments	6	.08	3.0	.81	19	.03	1.15	.03												
Location	Seta River				Yabase				Shina				Katada				Okinoshima			
Sort of Fish	N	min	max	ave	N	min	max	ave	N	min	max	ave	N	min	max	ave	N	min	max	ave
Carassius auratus gradoculis	5	1	3	2	5	1	3	2	14	.2	1	.5	5	.4	1	.7	5	.3	1	.6
Cyprinus carpio					5	.2	.8	.5												
Gnathopogon elongatus					5	.4	.9	.6					5	1	2	1				
Plecoglossus altivelis					5	.9	1	1	5	.3	.4	.3	5	.3	2	1				
Zacco platypus	9	2	6	3					7	.7	2	2					9	.2	.5	.4
"Isagi"													9	.1	.2	.2	9	.1	.2	.2
Opsariichthys uncirostris													5	.4	.9	.6	5	.3	.7	.5
Crustoean	5	.7	1	.8	5	.6	.8	.7					10	.03	.2	.08	10	.04	.1	.08
Plankton					1			.9	2	.03	.05	.04	2	.02	.08	.05	2			.02
Algae	2	.09	.1	.1	1			.04												

Table 12 PCB Residues in Birds (ppm µg/g)

Location Tissue Common Name	Sample number	Epidermis	Fat	Muscle	Remarks
Grebe	1	78.4	144.0	2.6	Collected on 11/30/72 at Kinohama,Moriyama City
Grebe	1	73.6	140.8	1.5	
Night Heron	1	25.6	82.4	2.5	Collected on 12/5/72 at Kitae, Chuzu Town
Dusky Mallard	1	48.8	74.4	3.0	Collected on 10/5/72 at Kinohama,Moriyama City
Dusky Mallard	1	4.1	3.9	0.15	

The picture representing the situation for the Seta River - Lake Biwa region is complicated by the locations of pollution sources such as condenser and paper recycling factories. Furthermore, confusion is compounded since many other pollution sources existed such as urban waste water. Therefore, assesment of discharged amounts and dissipation into sediment material becomes complicated and analytical evaluation is rendered extremely difficult.

2.2.2 The Case in Tsuruga Bay (Figs. 5 and 6)

This bay faces the Japan Sea and is located in the central part of the Japanese archipelago on the main island of Honshu. It possesses horizontal dimensions of 5 kilometres in width and 10 kilometres in depth; furthermore, it is relatively deep since more than half of its area has a vertical depth exceeding 40 metres. At the head of the bay is situated Tsuruga Port which is protected from the open ocean by a breakwater. Inside the port, sedimentary materials consist primarily of sand and mud, whereas only sand is found in the inner part of the bay. Traversing from the central part to the mouth, the bottom topo-

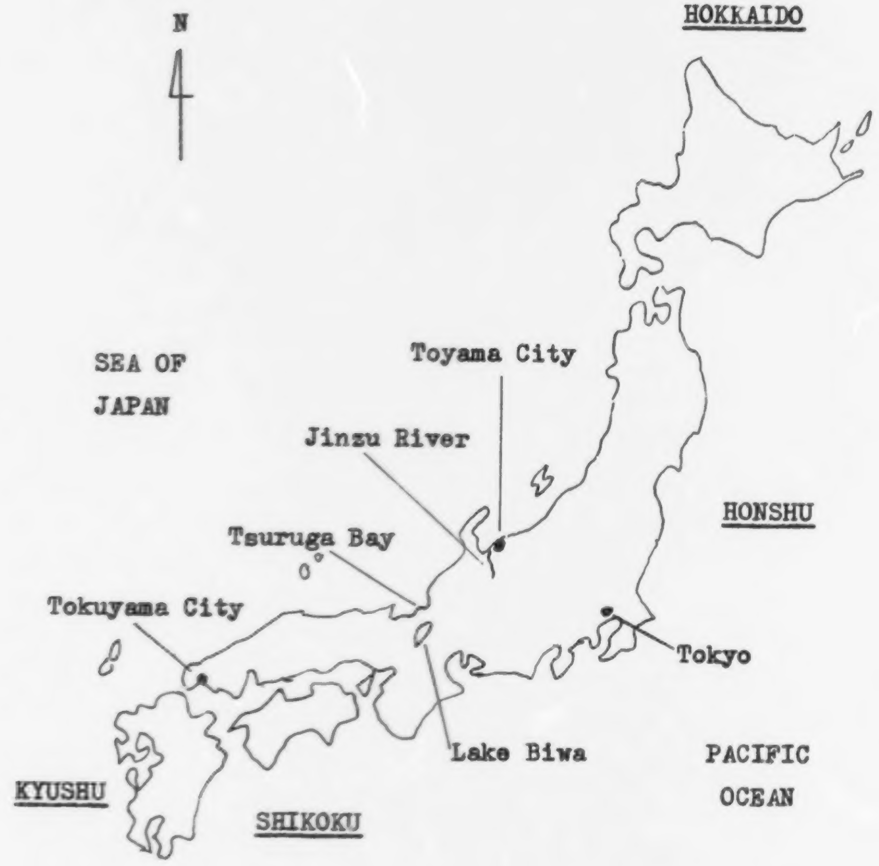

N
HOKKAIDO
SEA OF JAPAN
Toyama City
Jinzu River
HONSHU
Tsuruga Bay
Tokuyama City
Tokyo
Lake Biwa
PACIFIC OCEAN
KYUSHU
SHIKOKU

Fig. 5 Geographical Regions of Surveys in the Text

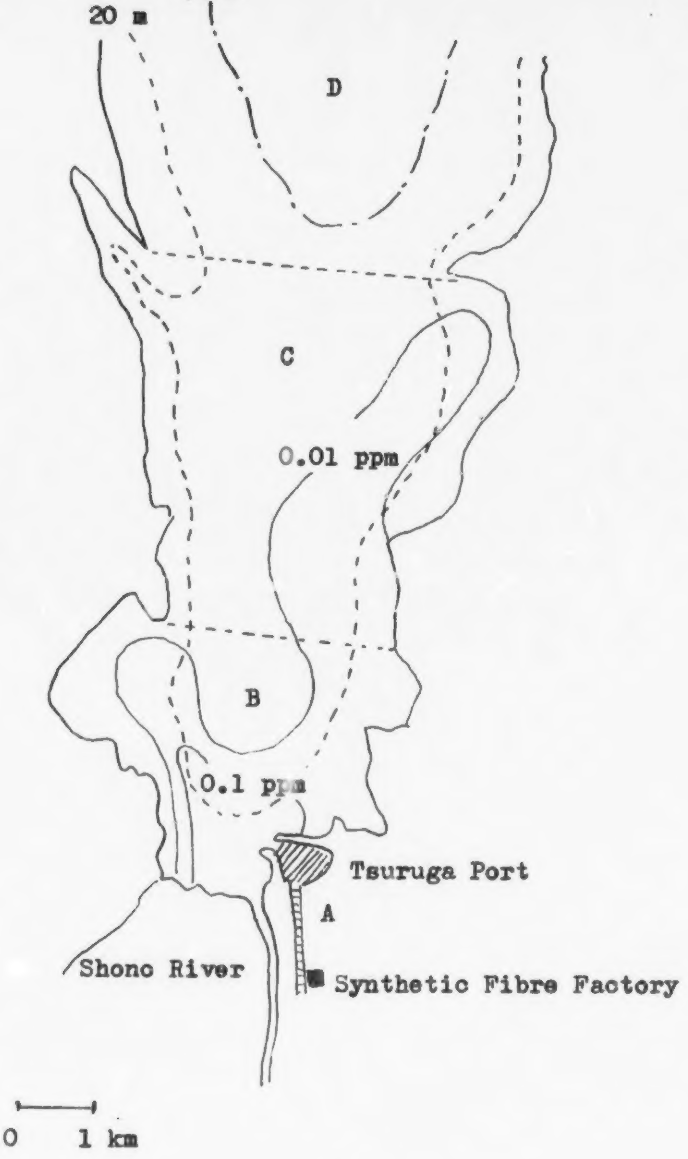

40 m
20 m
D
C
0.01 ppm
B
0.1 ppm
Tsuruga Port
A
Shono River
Synthetic Fibre Factory
0
1 km

Fig. 6 Map of Tsuruga Bay. PCB residues in sediments are indicated by hatched areas as follows: 20-200 ppm ▤ ; 1- 10 ppm ▨ .

graphy is characterised by the presence of pebbles and reefs. The shortline is precipitous except for that portion where Tsuruga City is situated.

Tsuruga City is relatively small with a population of about 60,000 inhabitants. Important local industries include commerce, synthetic fibres, cement and nickel alloy manufacture. Sheltered within the deep natural harbour provided by the bay, the city has become an important shipping centre on the western coast of Japan. Moreover, it also enjoys a prosperous fisheries industry which centres around Tsuruga fishing port; annual landing of fisheries products was 1,483 tons in 1973.

(1) Quantities of Utilisation and Discharge of PCBs

A certain plant located in this region which had engaged in the manufacture of synthetic fibres utilised KC 400 as a heat exchanger for more than 8 years from December, 1964 to June, 1973. During this period, a total of 469,220 kilograms was used of which 337 kilograms were suspected to have been discharged into the harbour via a canal.

(2) Environmental Levels (1973)

PCB levels detected in Tsuruga Bay are presented in Table 13 and indicate averages and ranges of residues found in water sediment, several species of fish and a single crustacean. Table 14 represents data reflecting concentrations of residues detected in blood of local inhabitants residing in the vicinity of Tsuruga City.

(3) PCB residues in bottom sediments

PCB pollution in Tsuruga Bay is considered to have originated mainly from a certain synthetic fibre factory located in Tsuruga City. On the basis of the demographic and industrial structure of the city, it is believed that sewage waste water which originated from households and public or private concerns did not contribute significantly to PCB pollution.

Waste water from said factory was discharged into Tsuruga Port via a canal. At the time when PCB discharge was suspended, concentrations of residues as shown in Fig. 6 were 20 to 200 ppm in mud taken from the canal and 1 to 10 ppm in sediments sampled from the port. The horizontal dissipation of PCB into the bay was found to be in the range of 0.1 ppm within a distance of 1 to 2 kilometres from the source, and 0.01 ppm at distances between 4 to 8 kilometres. Within 100 metres from the exit of the drainage conduit, distribution of PCB residues was determined and values are shown in Fig. 7.

Following suspension of PCB discharge from the factory, residues contained in sludge of the canal are believed to have dissipated into the bay together with suspended materials or dissolved in a free configuration in water. In either case, the sludge was suspected to have been the source of the ensuing and persistent PCB pollution. As a countermeasure intended to eliminate the source of contamination, operations to remove sludge from the canal were carried out on three occasions in 1973 during which approximately 11,200 cubic metres were removed. Residue analysis revealed that this operation resulted in a decrease in PCB concentration of canal water such that values were below 10 ppm. Since the average concentration of PCB residues detected in canal sediments was 90 ppm, and in view of the fact that water constituted 90% of the sludge, it was calculated that about 100 kilograms of PCBs were still present in the canal. This value corresponds to about one -third of the total discharged amount. As a matter of reference, the results of fractionation of sediments taken from the bay into grain-size bands which correspond to precipitation time according to the Atterberg method together with values of PCB residues are presented in Table 13. From these results, it is believed that PCB becomes adsorbed onto suspended grains which have diametres less than 2 microns and require greater than 24 hours for precipitation. This suggests that considerable attention should be given to the role which such minute suspensions play in environmental pollution.

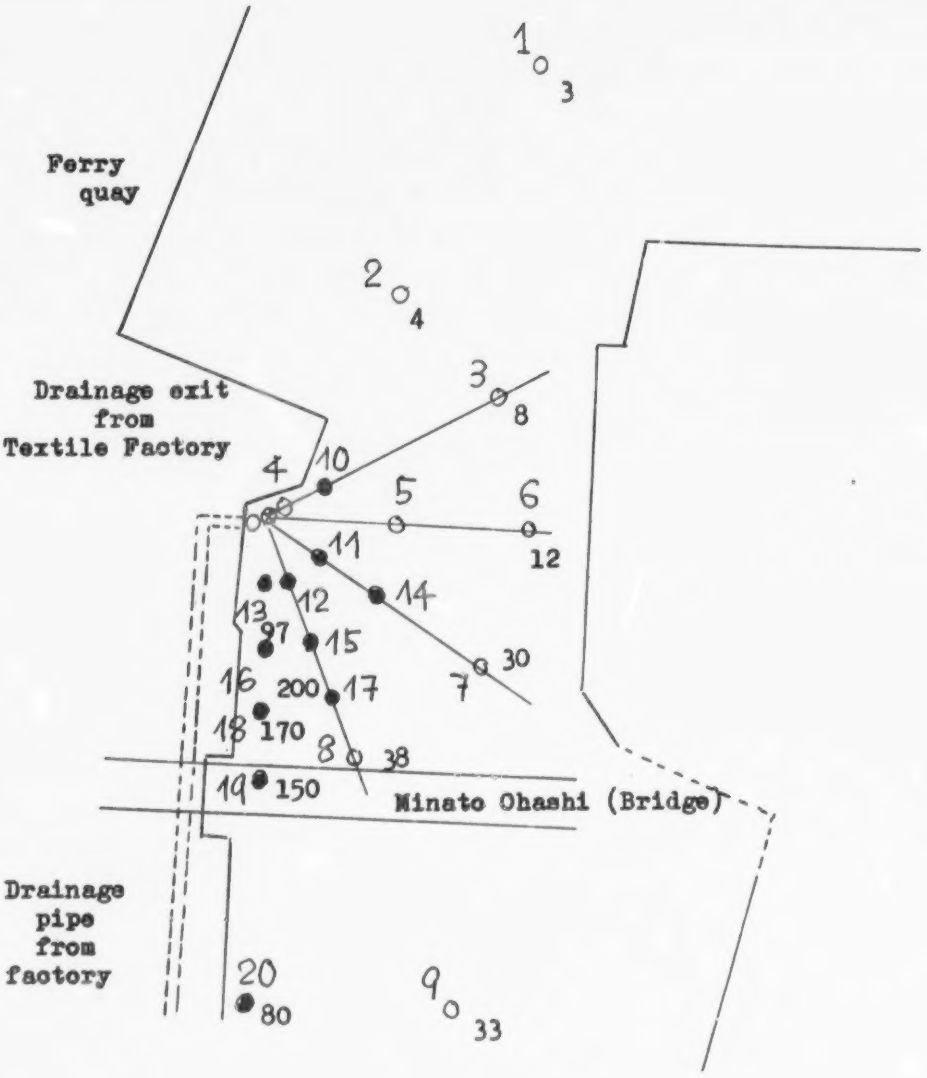

Ferry quay
1 3
2 4
3 8
Drainage exit from Textile Factory
4
10
5
6 12
11
14
13
12
97
15
7 30
16
200
17
18 170
8 38
19 150
Minato Ohashi (Bridge)
Drainage pipe from factory
20 80
9 33

Fig. 7 Results of a survey of PCB residues detected in sediments in the vicinity of the drainage outlet of a certain textile factory. Sampling dates indicated by (○) 12 /25/72; (●) 1/25/73; (⊗) 3/16/73. Large numerals from 1 to 20 indicates sampling locations; small numerals from 3 to 200 indicated concentration of PCB residues in sediments in ppm.

As a consequence in view of the aforementioned results, it was concluded that the distribution of PCB residues present in sediment of the bay resulted from the deposition of suspended materials which was influenced by the movement of water flowing into the bay from the Shono River as well as tidal effects.

Table 13 The Distribution of PCB residues According to Particle Size of Sediments from Tsuruga Bay

Particle Size	Fr A	Fr B	Fr C	Fr D	Fr E	Fr F
Sedimentation Time (hr)	0.5	0.5- 1	1 -3	3- 12	12 - 24	24
Composition (%)	63.6	5.5	15.1	11.1	2.5	2.3
PCB Residues*** (μg/g dry wt)						
1st analysis	2.9	3.2	3.2	4.1	5.0	-**
2nd analysis	2.7	3.9	3.4	3.9	5.9	6.1
3rd analysis	2.1	3.9	3.2	2.1	5.6	-
Average value	2.6	3.7	3.3	4.0*	5.5	6.1

* Value for 3rd analysis not included

** Not analysed

*** Standard KC-400

Fourth US/JAPAN Conference
on
Sewage Treatment Technology
Paper No. 5

CASE STUDIES OF COMPREHENSIVE WATER QUALITY MANAGEMENT PLANNING

October 29, 1975
Washington, D. C.

Ministry of Construction
Japanese Government

CASE STUDIES OF COMPREHENSIVE WATER QUALITY MANAGEMENT PLANNING

Satoru Tohyama, Head, Sewage Works Division, Department of Sewerage & Sewage Purification, Ministry of Construction

Page
1. Characteristics of the Planning Area 355
2. Environmental Water Quality Standards and the Target Periods 356
3. Present State of Water Qualities 357
4. Pollutant Loads According to the Classified Sources Present and Future 357
5. Forecast of Future Water Qualities 362
6. Allowable Pollutant Loads and Pollutant Loads to be Removed of the Respective Phases 365
7. Setting of Sewage Service Areas 366
8. Sewerage Study Program 369
9. Proposed Sewerage Service Area of Each Municipalities for Respective Phases 373
10. Effects of the Program on Water Quality Protection 374
11. Future Studies 375
12. Implementation of Sewage Works 375

CASE STUDIES OF COMPREHENSIVE WATER QUALITY MANAGEMENT PLANNING

COMPREHENSIVE OHTA AND SENO RIVER BASIN SEWERAGE STUDY PROGRAM

1. CHARACTERISTICS OF THE PLANNING AREA

Ohta and Seno Rivers are the largest rivers in Hiroshima Prefecture which run into Hiroshima City and their catchment area is about 21% of Hiroshima Prefecture.

From the Ohta River, we get about 700,000 m^3/day for water supply in the Metropolitan Hiroshima. Furthermore it will be expected that we will get more in the future because it is estimated to increase to 1.5 million m^3/day of water consumption. Their catchment area is about 1,810 km^2 ; the upper river basin is generally covered with broadleaved trees, the middle river basin is covered with coniferous trees and both mountaneous areas are over 80% of the catchment area. The lower river basin extends radially into a plain with center around the delta at the estuary of the Ohta River and surrounded by mountains and hills of 200 to 400 m high.

Hiroshima City thrives on the plain, from which the Metropolitan Hiroshima is extending to serve as a nerve center of the Chugoku and Shikoku Regions and taking a leading part in socio-economic activities.

Chugoku
Hiroshima
Kinki
Shikoku
Kyushu

Based on this backdrop, the basin has been pushing forward its urbanization rapidly in recent years with Hiroshima City as the hub.

In 1970, the population was 930 thousand and the industrial outputs amounted to about 550 thousand million Yen. With the concentration of the population and the industries, the urban area of the Metropolitan Hiroshima is spread greater and greater and it is forecast that in 1990 the population will increase to 1.48 million persons and the industrial outputs will be gained about 2,380 thousand million Yen. Our concern is more serious water pollution which will be caused by increased quantity and aggravated quality of both domestic sewage and industrial wastewater to be generated by inflated city activities.

In fact, there are many who have been clamouring for drastic measures against water pollution and it will be high time to prepare and implement a comprehensive river basin sewerage study program.

2. ENVIRONMENTAL WATER QUALITY STANDARDS AND THE TARGET PERIODS

Types of waters according to the environmental standards concerning water quality for these basin and the target periods for attaining the required water quality are designated as follows:

Table 1 Environmental Water Quality Standards in Ohta and Seno Rivers

Waterways	Category	Target	Temporary target
Ohta River (upper reaches)	A	1970	
Ohta River (lower reaches)	B	1970	
Enko River	C	1980	D (1975)
Original Ohta River	A	1970	
Temma River	A	1970	
Motoyasu River	A	1970	
Kyobashi River	A	1970	
Seno River	B	1980	C (1975)

Note: 1. This standards were applied at the cabinet council in 1970.
2. A, B, C and D are the water quality standards of a waterways according to the water use (Table 2).

Table 2 Environmental Water Quality Standards in Rivers

Item / Category	Standard values				
	pH	Biochemical oxygen demand (BOD)	Suspended solid (SS)	Dissolved oxygen (DO)	No. of colitis germs groups
AA	6.5 ~ 8.5	1 ppm or less	25 ppm or less	7.5 ppm or more	50 Mpn/100 mℓ or less
A	6.5 ~ 8.5	2 ppm or less	25 ppm or less	7.5 ppm or more	1,000 Mpn/100 mℓ or less
B	6.5 ~ 8.5	3 ppm or less	25 ppm or less	5 ppm or more	5,000 Mpn/100 mℓ or less
C	6.5 ~ 8.5	5 ppm or less	50 ppm or less	2 ppm or more	
D	6.0 ~ 8.5	8 ppm or less	100 ppm or less	2 ppm or more	
E	6.0 ~ 8.5	10 ppm or less	Floating matter such as garbage should not be observed.	2 ppm or more	

In order to accomplish the designated environmental water quality standards, it is necessary to select representative stations for respective waterways and evaluate water quality based on the data obtained therefrom. Also, these representative stations will be useful in the effect assessment and the review of the program implementation.

For these reasons, eight environmental representative station points and two additional points in the upper reaches of the Ohta River were established.

These ten stations are located as shown in Fig. 2.

(Of them, double-encircled points denote the standard stations of environmental water quality standards.)

3. PRESENT STATE OF WATER QUALITIES

Table 3 Present State of Water Qualities at 10 Stations (in 1970)

Station No.	Waterways	BOD	Environmental water quality standards (BOD)
		ppm	ppm
9	Ohta River	1.0	2
10	Ohta River	1.4	2
1	Ohta River	1.6	2
2	Ohta River	4.5	3
3	Temma River	1.3	2
4	Original Ohta River	1.2	2
5	Motoyasu River	0.6	2
6	Kyobashi River	1.7	2
7	Enko River	10.1	8 (5, by 1980)
8	Seno River	3.2	5 (3, by 1980)

4. POLLUTANT LOADS ACCORDING TO THE CLASSIFIED SOURCES – PRESENT AND FUTURE

The pollutant loads generated in the projected areas are roughly classified into three; those due to domestic sewage, those due to industrial wastewater and those due to discharges from livestock farmings. Pollutant loads from each source are estimated using the unit standard values such as per capita pollutant load.

As an elementary factor of estimation, population, industrial output and the number of heads of livestock are applied for a domestic sewage, an industrial wastewater and a discharge from a livestock farmings, respectively. Thus, each pollutant loads are calculated from the product of the elementary factor and the unit standard value.

The program term is 1975, 1980, 1985 and 1990, as taking account of the accomplishment of the water quality standard of the waterways.

(1) Elementary factor of estimation

The elementary factor for respective local governments district for respective phases are estimated with concerning the frame values appearing in the Hiroshima Prefectural Government's Long-term Comprehensive Plan whose target term is set at 1985 as the high level plan.

a) Population

On the basis of the Prefectural Government's Long-term Plan for the year 1985 and in consideration of the existing state of the development of this area, the population of each local community for 1985 is determined and the populations of local communities for respective phases are estimated by using a trend method from the past populations.

b) Industrial output

Based on the values for 1975 and 1985 appearing in the Prefectural Government's Long-term Plan, each local community's values are estimated by the product of the existing and planned areas for industries employing 30 persons or more and the output per unit area. And the final estimated each local community's outputs for 1975 and 1985 are revised by the ratio of the outputs by industries of 30 persons or more to the total outputs for 1970. Then, the outputs for the respective phases are estimated by using a trend method from the past industrial outputs.

c) Livestock farming

As the stock rearing is negligibly small in except for a pig rearing in this river basin, the number of pigs alone is estimated here.

As same as other factors, based on the Prefectural Government's Long-term Plan, the population of pigs is estimated for respective phases, by using a trend method from the past data.

Legend

↔	Stretch of designated type of the environmental water quality standards
▬	Catchment area
—·—·—	Municipal boundary line
○	Standard stations of the environmental water quality standards

Name of standard stations of the environmental water quality standards

9	Confluence of the Daimoji River (Ohta River)
10	Kumura (Ohta River)
1	Intake point of the Hesaka Water Supply (Ohta River)
2	Asahi Br. (Ohta canal)
3	Showa Great Br. (Temma River)
4	Funairi Br. (Original Ohta River)
5	Minami Great Br. (Motoyasu River)
6	Miyuki Br. (Kyobashi River)
7	Niho Br. (Enko River)
8	Hiura Br. (Seno River)

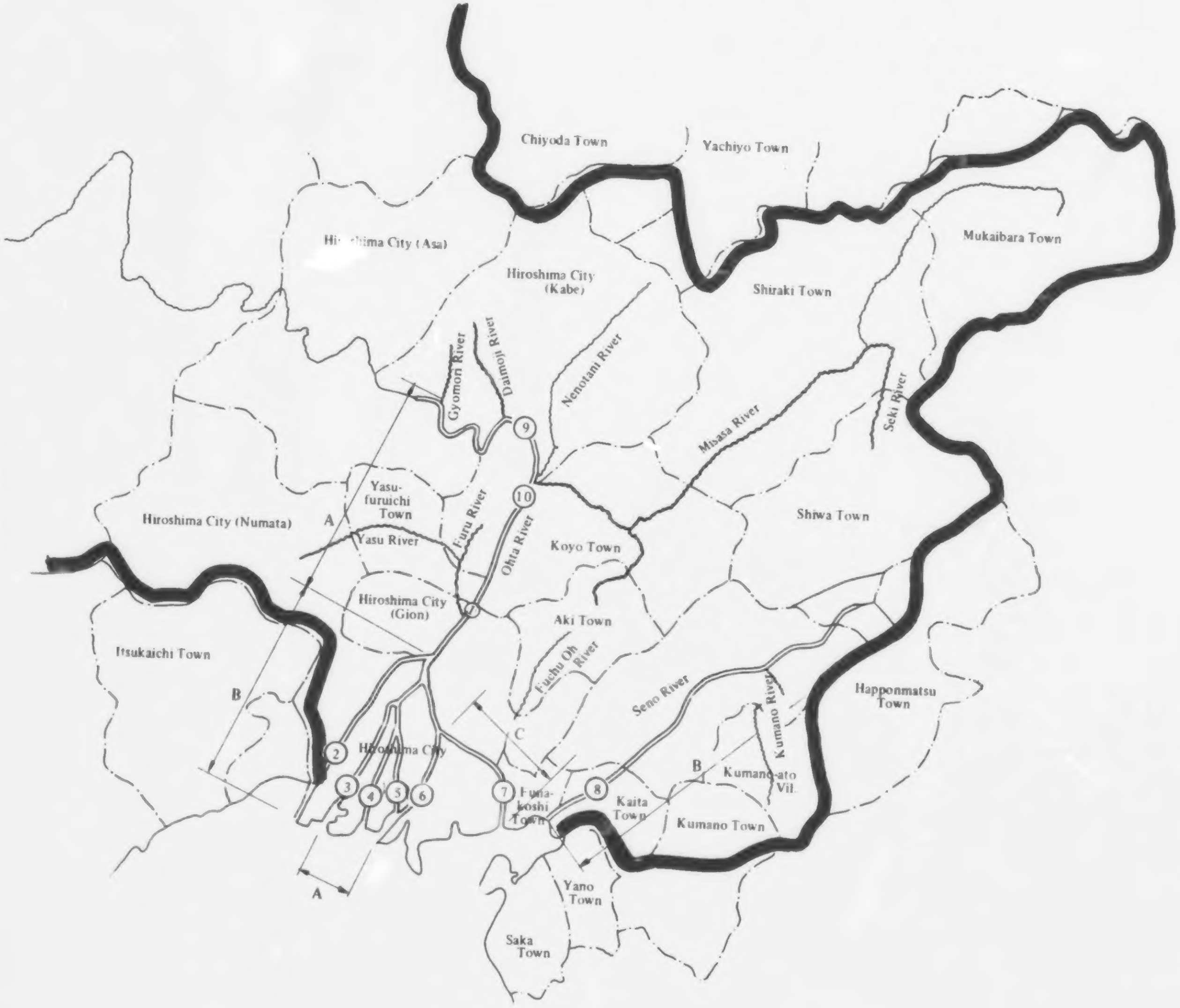

Chiyoda Town
Yachiyo Town
Mukaibara Town
Hiroshima City (Asa)
Hiroshima City (Kabe)
Shiraki Town
Gyomon River
Daimoji River
Nenotani River
Misasa River
Seki River
Yasu-furuichi Town
Hiroshima City (Numata)
A
Yasu River
Furu River
Ohta River
Koyo Town
Shiwa Town
Hiroshima City (Gion)
Aki Town
Itsukaichi Town
Fuchu Oh River
Seno River
Happonmatsu Town
B
C
Kumano River
Hiroshima City
Funa-koshi Town
Kaita Town
Kumano-ato Vil.
Kumano Town
Yano Town
Saka Town
1
2
3
4
5
6
7
8
9
10

Fig. 2 Location Map of Standard Stations of the Environmental Water Quality Standards

(2) The unit standard value for water quantity

a) Domestic sewage

Based on the water supply in 1985 appearing in the Prefectural Government's Long-term Comprehensive Plan, a quantity per capita is estimated for 1970 and a unit standard value of quantity for a domestic sewage is determined from the sum total of a quantity per capita, a commercial water consumption and a ground water.

Table 4 Unit Standard Values of Domestic Sewage for Respective Phases

(ℓ/capita/day)

Local Gov.	1970	1975	1980	1985	1990
Hiroshima City	500	600	700	800	900
Fuchu Town	350	450	550	650	750
Funakoshi Town					
Kaita Town					
Itsukaichi Town					
Gion (Hiroshima City)					
Yasufuruichi Town					
Sato Town					
Kabe (Hiroshima City)					
Koyo Town	200	300	400	500	600
Numata (Hiroshima City)					
Yano Town					
Saka Town					
Aki Town					
Senogawa Town					
Others					

b) Industrial wastewater

Industry-wise specific units of water consumption for each areas in 1965 and 1970 are taken as a basis for the estimation of a unit standard values.

c) Unit standard values of quantity for pigs.

Table 5 Unit Standard Values of Quantity for Pigs

(ℓ/head/day)

Animal	Standard value of sewage
Pig	13.5
Others	0

(3) Unit standard values of water quality

a) Unit standard values of quality for domestic sewage.

Table 6 Unit Standard Values of Quality for Domestic Sewage

(g/capita/day)

Constituent	1970	1975	1980	1985	1990
BOD	45	53	60	67	74
SS	41	48	54	61	67

b) Unit standard values of quality for industrial wastewater

More than 90% of industrial waste water is accounted for by the effluents from those factories which are discharging 50 m³ a day or more and then unit standard values for industrial wastewater are estimated from notified data and Environmental Protection Agency's data.

Table 7 Unit Standard Values of Quality for Industrial Wastewater

(ppm)

Trade	BOD	COD	SS
Food processing	325	105	60
Textile	10	10	100
Clothing	10	10	30
Lumbering and wood-working	10	10	30
Furniture and furnishings	10	10	40
Pulp and paper-milling	35	25	240
Printing and publishing	10	10	30
Chemical	100	80	125
Petroleum and coal	80	40	120
Rubber	10	20	50
Tanning and leather processing	10	10	20
Ceramic, maisonry, etc.	10	50	90
Iron and steel	20	85	140
Nonferrous metals	20	27	105
Metals	10	40	220
General metals	15	6	10
Electric machinery	10	15	5
Transport machinery	20	20	15
Precision tools	10	10	60
Others	20	10	20

c) Unit standard values of quality for a pig

Table 8 Unit Standard Values of Quality for a Pig

(g/head/day)

Animal	BOD	SS
Pig	165	770
Others	0	0

5. FORECAST OF FUTURE WATER QUALITIES

The forecast of future qualities of the upper reaches of the Ohta River and the Seno River are estimated by an ordinary method of dividing the reached pollution loads to waterways by water flow. But the changes in the flow velocity due to change in tidal level are taken into account for the assessment of water qualities in the tidal rivers in Hiroshima City since as large a tidal level change as some 3 m denies the computation of water qualities from the water flow rate only.

(1) Existing hydrographic conditions and future plan

The 15-year mean hydrographic conditions observed at station (10) (Kumura point of the Ohta River) for the period of 1954 to 1968 are shown below and the low water flow is 35.2 m^3/sec.

Table 9 Hydrographic Conditions of Ohta River System at Station 10 (Kumura)

(Mean value for 1954 ~ 68 period, m^3/sec.)

High water flow (m^3/sec.)	Ordinary water flow (m^3/sec.)	Low water flow (m^3/sec.)	Doughty water flow (m^3/sec.)	Minimum water flow (m^3/sec.)	Yearly average water flow (m^3/sec.)	Yearly total runoff ($10^6 m^3$/yr.)	Catchment area (km^2)
79.60	49.40	35.20	21.00	12.60	75.20	2,371.6	1,481.0

The standard low water flow at St. (1) (intake point of the Hesaka Water Supply) is assumed about 26.0 m^3/sec., but will be reviewed if the plans for the construction of dams in the upper reaches are finalized.

With regard to the Seno River, there is no hydrographic data. Therefore, based on the flow conditions at the upper dam sites, precipitation conditions and catchment area, the low water flow at St. 8 (Hiura Br.) is estimated as 0.57 m^3/sec.

Table 10 Hydrographic Conditions of the Seno River at Station 8 (Hinoura Br.)

(Mean value for 1959 ~ 68, m^3/sec.)

High water flow (m^3/sec.)	Ordinary water flow (m^3/sec.)	Low water flow (m^3/sec.)	Doughty water flow (m^3/sec.)	Minimum water flow (m^3/sec.)	Yearly average water flow (m^3/sec.)	Yearly total runoff ($10^6 m^3$/yr.)	Catchment area (km^2)
2.08	1.00	0.57	0.23	0.11	2.28	71.99	106.3

Table 11 Population, Output and Number of Heads of Livestock

1) Persons
2) Million Yen
3) Heads

	1970			1975			1980			1985			1990		
	Population1)	Output2)	Livestock3)	Population	Output	Livestock	Population	Output	Livestock	Population	Output	Livestock	Population	Output	Livestock
Ohta River	852,730	531,008	6,770	985,500	1,074,780	7,220	1,112,900	1,328,983	7,650	1,237,900	1,814,613	7,990	1,351,100	2,176,614	8,450
Seno River	104,142	23,611	5,160	113,100	61,909	5,790	121,100	77,694	6,420	129,800	204,685	7,050	137,500	243,696	7,570
Total	956,872	554,619	11,930	1,098,600	1,136,689	13,010	1,234,000	1,406,677	14,070	1,367,700	2,019,298	15,040	1,488,600	2,420,310	16,030

Table 12 Quantity

(1,000 m³/day)

	1970			1975			1980			1985			1990		
	Domestic	Industrial	Total	Domestic	Industrial	Total	Domestic	Industrial	Total	Domestic	Industrial	Total	Domestic	Industrial	Total
Ohta River	367.8	92.8	460.6	509.4	180.4	689.8	678.7	289.7	918.4	870.2	317.7	1,187.9	1,083.2	372.7	1,455.9
Seno River	20.8	6.5	27.3	34.0	11.3	45.3	48.4	12.2	60.6	65.4	26.4	91.8	82.6	31.4	114.0
Total	388.6	99.3	487.9	543.4	191.7	735.1	727.1	251.9	979.0	935.6	344.1	1,279.7	1,165.8	404.1	1,569.9

Table 13 Pollutant Loads

(kg/day)

	1970				1975				1980				1985				1990			
	Domestic	Industrial	Livestock	Total	Domestic	Industrial	Livestock	Total	Domestic	Industrial	Livestock	Total	Domestic	Industrial	Livestock	Total	Domestic	Industrial	Livestock	Total
Ohta River	38,373	9,388	1,112	48,873	52,231	17,879	1,187	71,297	66,774	20,930	1,258	88,962	82,879	38,382	1,314	122,575	99,983	43,997	1,394	145,374
Seno River	4,688	141	846	5,675	5,643	376	949	6,968	7,266	446	1,053	8,765	8,363	1,593	1,158	11,114	10,174	1,813	1,242	13,229
Total	43,061	9,529	1,958	54,548	57,874	18,255	2,136	78,265	74,040	21,376	2,311	97,727	91,242	39,975	2,472	133,689	110,157	45,810	2,636	158,603

(2) Reached coefficient and reached pollutant loads

a) Reached coefficient

Table 14 Reached Coefficient

Classification			Reached coefficient
Sewer-service area			1.0
Except sewer-service area	Miscellaneous discharge	Urban	0.9
		Rural	0.2
	Night soil	Night soil treatment plant	0.2
		Septic tank	0.35
		Reclamation to farm-land	0.1
	Livestock sewage	Urban	1.0
		Rural	0.4
	Industrial wastewater		1.0

b) Reached pollutant loads

The reached coefficients listed in Table 15 are used and the upper limit of unit standard value for industrial wastewater is set at BOD 120 ppm for all trades. The quality of the final effluents from the sewage treatment plant is set at 20 ppm.

Based on these conditions, reached pollutant loads are calculated and distributed against the standard station of water quality standards.

(3) Forecast of water qualities and its methodology

The calculation of water qualities is carried out with respect to the Ohta River's upper reaches and the Seno River on one hand and to tidal waterways in Hiroshima City on the other.

a) Ohta River's upper reaches and Seno River

For this area, the following equation is applied.

$$L_A = L_B 10^{-kt} + L_{B'}$$

$$S = L_A / Q$$

Where,

L_B : Pollutant loads in the upper reaches (gr/day)
K : Self-purification coefficient (ℓ/day)
t : Reaching time (days)
$L_{B'}$: Pollutant loads directly flowing out to the lower reaches without passing through rivers having self-purification function (gr/day)
L_A : Pollutant loads in the lower reaches (gr/day)
S : Water quality (ppm)
Q : River flow (m^3/day)

Prior to computation, the existing water qualities are applied to the above equation in order to determine the self-purification coefficient, K.

The results are as follows.

Upper reaches of Ohta River:	$K = 2.0$ (1/day)
Seno River:	$K = 0.8$ (1/day)

b) Tidal waterways in Hiroshima City

For this area, the following continuity equation for pollutant taking account of the change in flow velocity due to tidal change is applied to the water quality computation.

$$S = \frac{L}{2d\sqrt{\pi Kux}}$$

Where, S : Water quality (ppm)
L : Reached pollutant load (kg/hr.)
d : Thickness of layer over which pollutants are included (m)
K : Diffusion coefficient (m^2/hr.)
u : Mean flow velocity within layer, d (m/hr.)
x : Distance (m)

For the purpose of computation, the observation data around the clock on the water quality vs. flow velocity relationship obtained by the Ministry of Construction in 1971 are applied to the above equation to determine the diffusion coefficient, K. The results are as follows.

Tidal waterways in Hiroshima City: $K = 360$ m^2/hr.

Now, representative water quality is so set as to meet the water quality standards for three quarters of a year corresponding to them at the low water flow. A day is divided into 8 phases of 3-hr. zones for computation and the second largest value is taken as representative.

6. ALLOWABLE POLLUTANT LOADS AND POLLUTANT LOADS TO BE REMOVED AT THE RESPECTIVE PHASES

The pollutant loads to be removed for the purpose of attaining the environmental water quality standards are calculated in the following:

First, the allowable pollutant loads for respective standard stations are calculated and are subtracted from the total reached pollutant loads. The difference is the load to be removed.

(1) Determination of the load to be removed by sewage works

The pollutant loads to be removed by the sewage works are calculated by using a ratio of the load due to the domestic sewage and the load due to the industrial wastewater.

(2) The determination of the loads of industrial wastewater to be removed by the effluents standard

The loads of industrial wastewater to be removed as determined under item (1) above are appropriated according to the effluents standards.

For example, trying to estimate the more stringent standard in order to attain the water quality standard as regards the Niho Bridge Station of the Enko River where the load reduction of industrial wastewater is the most largest, the average quality of the industrial wastewater is to be limited to within as tight a value as 20% in 1980 and it is very stringent.

A control method for industrial wastewater should be prepared as attaining the water pollution control in consideration of the prospects of sewage works, beneficiaries shares, usage costs system according to the quality, etc.

7. SETTING OF SEWAGE SERVICE AREAS

(1) Setting of unit areas

One city and eleven towns are divided into 26 unit areas according principally to the administrative division and in consideration of geographical conditions as well as coordination with the tributary areas of the standard stations. See the table below.

Table 15 Number of the Unit Areas and Names of Administrative Areas

Unit No.	Name of administrative area	Unit No.	Name of administrative area	Unit No.	Name of administrative area
1	Aki Town	10	Hiroshima City (Kogo)	16′	Hiroshima City (Nakayama)
2	Koyo Town	11	Itsukaichi Town	17	Fuchu Town
3	Hiroshima City (Kabe)	12	Hiroshima City (Eba)	18	Senogawa Town
4	Sato Town	13	Hiroshima City (Senda)	19	Kaita Town
5	Koyo Town	14	Hiroshima City (Asahimachi)	20	Saka Town
6	Hiroshima City (Hesaka)	14′	Hiroshima City (Shinonome)	21	Yano Town
7	Hiroshima City (Numata)	15	Hiroshima City (Oozu)	22	Funakoshi Town
8	Yasufuruichi Town	15′	Hiroshima City (Ushita)	23	Hiroshima City (Aosaki)
9	Hiroshima City (Gion)	16	Aki Town		

(2) Setting of proposed sewage service areas

A proposed sewage service area is principally the urbanization designated areas for 1980 and the areas which are projected as urban areas according to the availability of residential areas, proposed development conditions and future population, etc. for 1990.

For the basin under consideration, the urban area is dotted with almost all of industries, and then, the industrial wastewater will be taken into the sewer system for the purpose of environmental protection of urban areas.

Table 16 Allowable Pollution Loads and Pollution Loads to be Removed

(BOD kg/day)

Items	1970				1975				1980				1985				1990			
	Domestic	Industrial	Others	Total	Domestic	Industrial	Others	Total	Domestic	Industrial	Others	Total	Domestic	Industrial	Others	Total	Domestic	Industrial	Others	Total
Generated loads	43,061	9,529	1,958	54,548	57,874	18,255	2,136	78,265	74,040	21,376	2,311	97,727	91,242	39,975	2,472	133,689	110,157	45,810	2,636	158,603
Reached loads	17,804	5,789	854	24,447	25,917	6,115	959	32,991	34,373	7,959	982	43,314	43,771	11,040	1,059	55,870	53,789	12,935	1,156	67,880
Allowable loads					17,054	3,703	959	21,716	15,643	3,341	982	19,966	15,420	3,765	1,059	20,244	15,800	3,630	6,156	20,586
Loads to be removed					9,471	2,414	–	11,885	18,730	4,618	–	23,348	28,351	7,275	–	35,626	37,989	9,305	–	47,294

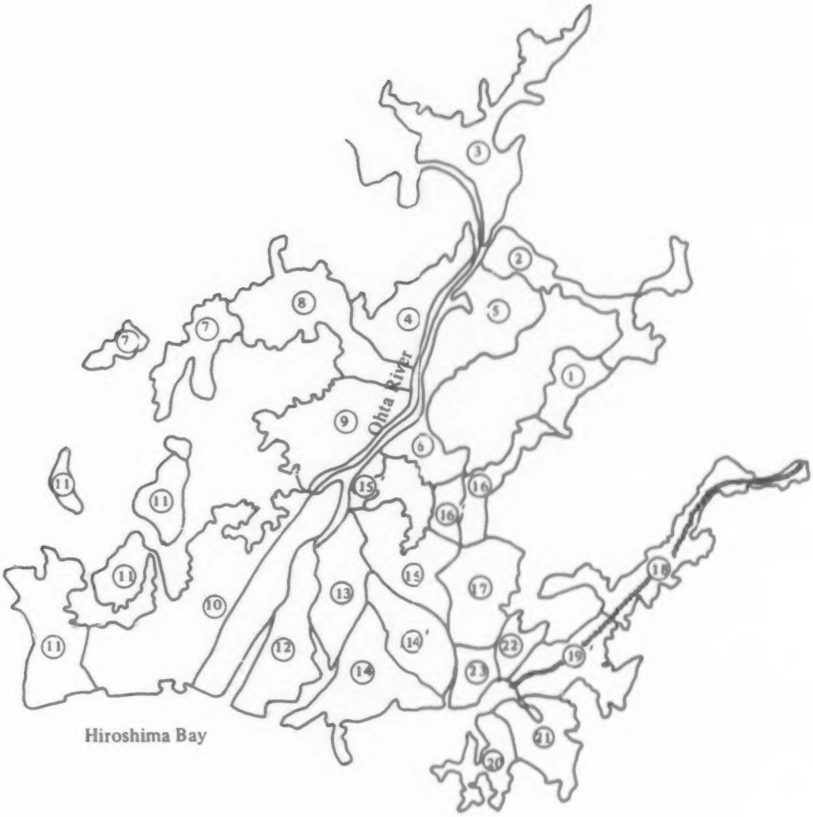
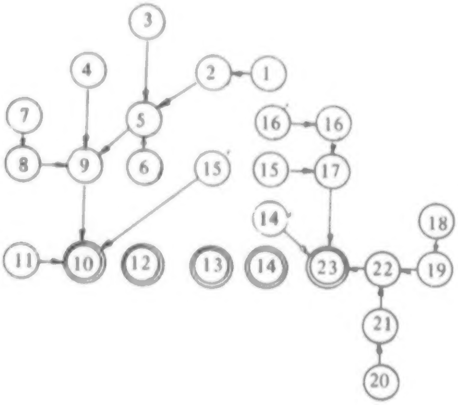

③
②
⑧
⑦
④
⑤
⑦
Ohta River
①
⑨
⑥
⑪
⑮
⑯
⑪
⑯
⑱
⑪
⑮
⑰
⑩
⑬
⑪
⑭'
㉒
⑫
⑲
⑭
㉓
Hiroshima Bay
㉑
⑳

Fig. 3 Number of the Unit Areas

8. SEWERAGE STUDY PROGRAM

(1) Optimum sewerage system and the reason of the decision

Studies are evaluated about total costs, existing sewage service, effects on the water quality, prospect of the aquisition of the treatment plant's site etc., with regard to a sewarage system to install a treatment plant in each unit and several systems in which are allocated a treatment plant per a few units and the optimum sewerage system is determined as follows:



(2) Plan of the sewerage facilities

The design criteria of the optimum sewerage system according to item 8-(1) is shown in Table 17. As will be clear from Table 21 showing a prediction of future water qualities, it is suggested that the water qualities of all points expect the Motoyasu River and the Kyobashi River will exceed the environmental standard values, particularly in the Enko River where pollution will be aggravated seriously.

Implementation of the program should therefore attach to the immediate effects on the water quality and the environment; namely, it is important to launch into the comprehensive sewage works as early as possible and to complete the local sewage works under way as early as possible, particularly the construction of a sewage treatment plant in Ohzu Area.

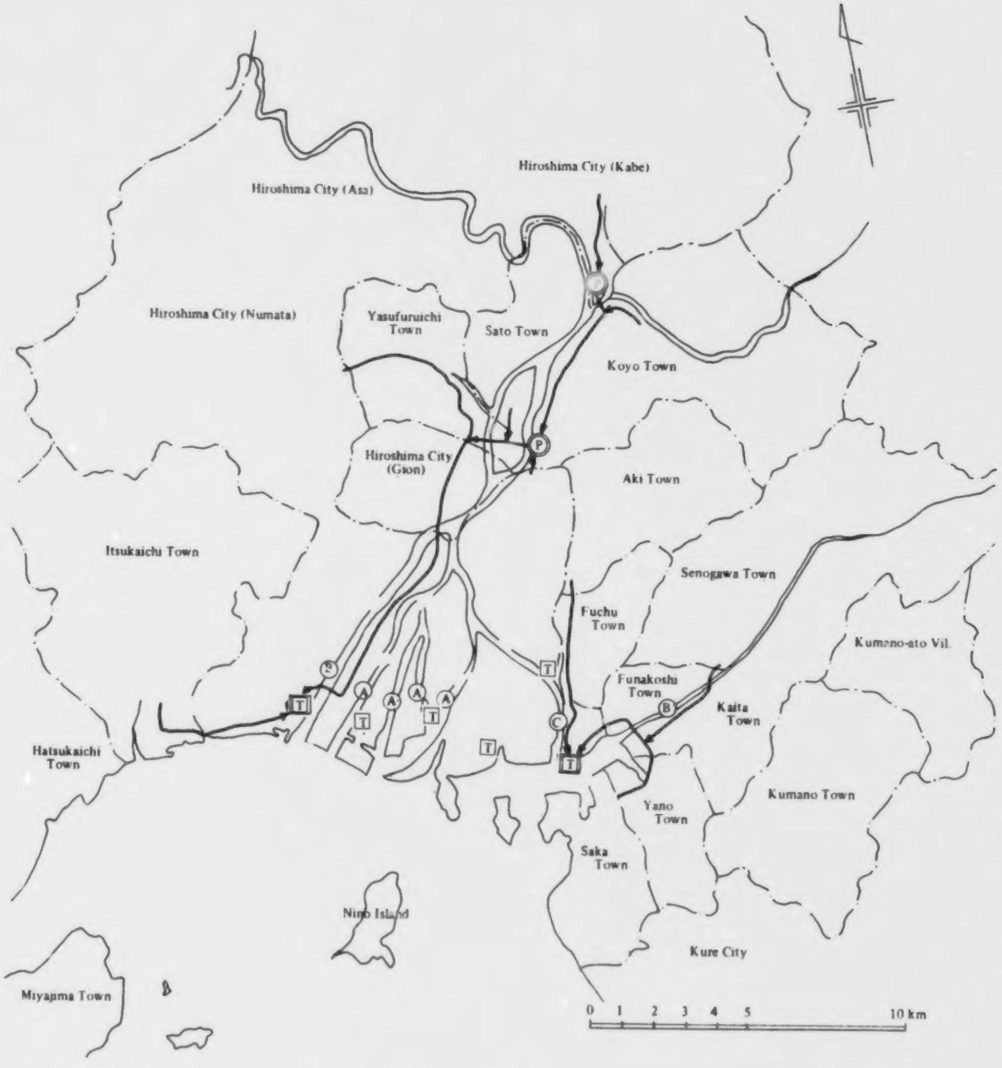
The design criteria are shown in Tables 17 and 18.

Table 17 Outline of the Design Criteria

Classification		Proposed service area (ha)	Proposed service population (thousand persons)	Proposed sewage quantity (m^3/day)	Number of sewage treatment plant and the area of plant's site required (ha)	Number of pumping stations	Sewer pipe diameter and alignment length (m)
Hiroshima City local sewage works	Senda service area	510	78.2	107,700	1/4.14	4	⊙ 0.25 ~ ⊡ 2.10 x 1.89 100,956
	Eba service area	616	90.1	98,300	1/2.91	4	⊙ 0.20 ~ ⊡ 2.30 x 2.07 198,790
	Asahi-machi service area	895	121.4	158,700	1/1.66	1	⊙ 0.20 ~ ⊡ 3.10 x 2.48 72,061
	Total	2,021	289.7	364,600	3/8.71	9	⊙ 0.20 ~ ⊡ 3.10 x 2.48 371,807
Ohta River and Seno River comprehensive sewage works	Ohta River service area	9,915	669.6	687,300	1/34.40	2	⊙ 0.70 ~ 3.00 39,900
	Seno River service area	4,660	354.3	393,600	1/35.00	–	⊙ 0.90 ~ 2.20 15,290
	Total	14,575	1,023.9	1,080,900	2/69.40	2	⊙ 0.70 ~ 3.00 55,190
Total		16,596	1,313.6	1,445,500	5/78.11		

Table 18 Outline of the Proposed Local Sewage Works in Each Municipalities

Name of service area	Names of municipalities	Service area (ha.)	Service population (thousand persons)	Sewage quantity (m^3/day)	Remarks
Ohta River service area	Hiroshima City	2,871	261.2	276,200	Hesaka, Kogo, Misasa, etc.
	Itsukaichi Town	1,305	69.3	90,300	
	Hiroshima City (Gion)	799	61.0	53,100	
	Yasufuruichi Town	1,030	65.9	54,400	
	Sato Town	603	37.6	30,500	
	Hiroshima City (Numata)	602	26.4	54,000	
	Hiroshima City (Kabe)	1,085	47.5	61,700	
	Koyo Town	1,340	88.2	58,600	
	Aki Town	280	12.5	8,500	
	Total	9,915	669.6	687,300	
Seno River service area	Hiroshima City	1,255	144.1	168,000	Ohzu, Shinonome, Nakayama, etc.
	Aki Town	321	19.7	13,500	
	Fuchu Town	634	51.0	68,500	
	Funakoshi Town	204	17.8	19,200	
	Kaita Town	605	39.3	44,400	
	Senogawa Town	756	32.0	29,800	
	Yano Town	520	33.0	29,600	
	Saka Town	364	17.4	20,600	
	Total	4,660	354.3	393,600	
Ohta River and Seno River comprehensive sewage works total		14,575	1,023.9	1,080,900	
Senda service area	Hiroshima City	510	78.2	107,700	
Eba service area	Hiroshima City	616	90.1	98,200	
Asahimachi service area	Hiroshima City	895	121.4	158,700	
Hiroshima City local sewage works total		2,021	289.7	364,600	
Total		16,596	1,313.6	1,445,500	


Hiroshima City (Kabe)
Hiroshima City (Asa)
Hiroshima City (Numata)
Yasufuruichi Town
Sato Town
Koyo Town
Hiroshima City (Gion)
Aki Town
Itsukaichi Town
Senogawa Town
Fuchu Town
Funakoshi Town
Kaita Town
Kumano-ato Vil.
Kumano Town
Yano Town
Saka Town
Kure City
Hatsukaichi Town
Nino Island
Miyajima Town
P
S
A
B
C
T
0 1 2 3 4 5 10 km

Legend

ⒶⒷⒸ : Water Quality Standards of Waterways

Ⓟ : Pumping Stations

T T : Sewage Treatment Plants

Fig. 4 Proposed Optimum Sewerage Plan

9. PROPOSED SEWERAGE SERVICE AREA OF EACH MUNICIPALITIES FOR RESPECTIVE PHASES

(1) Priority and its concept

The priority for sewage works of each municipalities is determined by taking into account the following:

a) Magnitude of effects on water quality

The order of the magnitudes of cost-benefit ratio (volume of removal of pollutant loads per unit cost) of the sewage works required to remove loads for the purpose of attaining the environmental standards at each stations in 1975.

b) Magnitude of necessity of improvement of the environment

The order of population densities (as census of 1970) reckoned as overcrowded index showing the urban environmental aggravation.

Based on the above two factors the conditions of sewage works under ways, conservation of water sources, aggravation of environment and other various factors are put together to determine the priority of sewage works as follows.

i) Areas to launch immediately into the sewage works

Hiroshima City (local sewage works, Ohta River Service Area, Seno River Service Area), Koyo Town, Hiroshima City (Gion), Yasufuruichi Town.

ii) Areas to launch as early as possible

Itsukaichi Town, Hiroshima City (Kabe), Sato Town, Fuchu Town, Funakoshi Town, Kaita Town, Senogawa Town, Yano Town.

iii) Areas to launch early

Aki Town, Hiroshima City (Numata), Saka Town.

(2) Proposed sewerage service areas for respective phases

The service area by the sewerage required for purpose of attaining the environmental water quality standards for 1975 and 1980 is shown in Table 19.

And for 1985 and 1990, sewerage service area will be covered to achieve pollutant load reductions more than specified in the guidelines for the purpose of improving amenity.

Table 19 Area of Coverage for the Planned Years

Year	1990 service area (ha)	Service area by year (ha)	Ratio of service area (%)
1975	16,596 (ha)	4,256	26
1980		7,931	48
1985		13,665	82
1990		16,596	100

10. EFFECTS OF THE PROGRAM ON WATER QUALITY PROTECTION

It is insufficient that the protection of the quality of public waterways presupposes at once consideration to the utilization of waters by men. We should take account into the conservation of minimum environmental standards necessary for fellow living creatures.

As regards the Ohta and Seno River Basin, the water quality should be restored to a point of allowing swimming as Hiroshima was once called as "the City of Waters".

Based on this principle, the program will be evaluated not only to improve the environment, consequently to prevent water pollution, to regain clean waters, and to conserve water sources, but also to improve the water quality to the level shown in Table 20 which will be high enough even if the environmental water quality standards are tightened in the future.

Table 20 Water Qualities at Each Stations for Respective Phases

Water qualities / Standard stations	Existing water quality (BOD ppm)	Classification	Predicted water quality (BOD ppm)			
			1975	1980	1985	1990
1. Ohta River, Intake point of Hesaka water supply	1.6 (A)	Left alone	2.9	4.4	6.0	7.6
		Improved by implementation	2.0	2.0	0.6	0.6
2. Ohta canal, Asahi Bridge	4.5 (B)	Left alone	5.7	7.3	9.0	10.9
		Improved by implementation	3.0	3.0	0.6	0.6
3. Temma River, Showa Great Bridge	1.3 (A)	Left alone	1.7	2.1	2.7	3.3
		Improved by implementation	1.7<	2.0	0.6	0.6
4. Original Ohta River, Funairi Bridge	1.2 (A)	Left alone	1.5	1.9	2.4	2.9
		Improved by implementation	1.5<	1.9<	0.6	0.6
5. Motoyasu River, Minami Great Bridge	0.6 (A)	Left alone	0.6	0.6	0.8	0.8
		Improved by implementation	0.6<	0.6<	0.8	0.8
6. Kyobashi River, Miyuki Bridge	1.7 (A)	Left alone	1.7	1.7	1.8	1.9
		Improved by implementation	1.7<	1.7<	1.6	1.7
7. Enko River, Niho Bridge	10.1 (C)	Left alone	12.0	14.8	18.2	21.7
		Improved by implementation	8.0	5.0	0.6	0.6
8. Seno River, Hinoura Bridge	3.2 (B)	Left alone	4.9	8.2	10.3	13.0
		Improved by implementation	4.9<	3.0	0.8	0.9

Note: Letters parenthesized denote types of waterways according to environmental water quality standards.

11. FUTURE STUDIES

In the preparation of the program, some assumptions are made for want of data concerning water pollution.

From now on, various data, especially those concerning water qualities should be collected in order to provide for every-five-year previous and the following studies should be pushed forward to refine the plans for promoting the sewage works reasonably.

(1) Improvement of the existing combined sewer system

The effects of wet weather conditions on the water quality of the combined sewer system should be studies in order to provide measures for sewage treatment and modification into separate sewer system and to determine their implementation schedule.

(2) Development of water pollution mechanisms in the tidal waterways

The tidal waterways in the lower reaches of the Ohta River (Hiroshima City) are experiencing some 3 m of tidal range. As pollutant loads running into the tidal waterways have different effects on water qualities depending on the tidal current and the change of water level, the pollution mechanisms should be developed along with analysis of measurements at stations with reference to the water quality standards.

(3) Examination of pollution in sea

Water quality conservation in the Bay of Hiroshima should also be studied with reference to the environmental water quality standards.

12. IMPLEMENTATION OF SEWAGE WORKS

According to the studies, the Ohta River comprehensive sewage works was launched in 1972 and a part of the alignment of trunk was completed in 1974.

From this year on, the construction of sewage treatment plant will be started.

Along with the Program, Hiroshima City local sewage works is also in progress.

Fourth US/JAPAN Conference
on
Sewage Treatment Technology
Paper No. 6

FURTHER DISCUSSIONS OF THE FEDERAL WATER POLLUTION CONTROL ACT OF 1972

October 29, 1975
Washington, D. C.

Ministry of Construction
Japanese Government

FUTURE DISCUSSIONS ON FEDERAL WATER POLLUTION CONTROL ACT AMENDMENTS OF 1972

T. Kubo, Japan Sewage Works Agency

1. INTRODUCTION 378
2. THE GOALS AND POLICY OF THE ACT 378
3. MUNICIPAL SECONDARY TREATMENT 379
4. FINANCING 381
5. OCEAN OUTFALLS AND OCEAN DUMPING 381
    5.1 Ocean Outfalls 381
    5.2 Ocean Dumping 382
6. PRETREATMENT 384
7. USER CHARGE 386
8. INDUSTRIAL COST RECOVERY 391

1. INTRODUCTION

At the Second U.S./Japan Conference on Sewage Treatment Technology, December 1 ~ 6, 1972, in Washington, D.C., presentation in relation to the Federal Water Pollution Control Act amendment of 1972 was made by U.S. side to the Japanese Delegation, when it was just beginning to implement the Act. During the Second Conference the Japanese Delegates recognized that the great amount of discussions had been made in the House and Senate on the Act itself and the national policy for environmental pollution, and the passage of the Act was really a historical event in the field of water pollution control in the United States. And the Japanese Delegates then paid attention on its thoroughness and comprehensiveness, and paid respects to the nation's ambitious goals which were set against water pollution control.

At the Third Conference, February 12 ~ 16, 1974, in Tokyo, Mr. Charles H. Sutfin of U.S. EPA Headquarters, Washington, D.C., described that since 1972 U.S. EPA had worked very hard, learned much and in doing so significant progress had been made toward full implementation of the Act. As we can see it in the Proceedings of the Third Conference that the state viewpoint and municipal viewpoint presented during the Third Conference made it clear that the various state governments and local municipalities had no single viewpoint on the merits of the 1972 Act. During the Third Conference various arguments and criticizms on the Act were raised by both the U.S. and Japanese side and also made some comments on it particularly from viewpoint of water quality standard, effluent standard, grants for construction of treatment works, research and related programs. It seems that most of state civil servants think that the Act has produced serious flaws in the field of water pollution control and has been wrongly implemented, for instance, Section 101 (b) of the Act states that it is the policy of the Congress to recognize, preserve and protect the primary responsibilities and rights of States to prevent, but actually the federal role is all pervasive and conformity to the federal mold is much more mandatory than statutory language would require, and the goals indicating recreational use water quality in all streams by 1983 and zero discharge of pollutants by 1985 are unrealistic, particularly the schedules. We realize that there must be difficult problems for full implementation of the Act, and nevertheless we would like to pay our respects to EPA's ambitious goals in the field of environmental pollution control through full implementation of the Act. Today, I would like to discuss with you on several points of the Act, but particularly to emphasize the specific area of pretreatment, user charge system and industrial cost recovery under the Act.

2. THE GOALS AND POLICY OF THE ACT

The goals and policy of the Act are declared in the Section 101 (a) of the Act, to wit:

(1) It is the national goal that the discharge of pollutants into the navigable waters be eliminated by 1985.

(2) It is the national goal that wherever attainable, an interium goal of water quality which provides for the protection and propagation of fish, shellfish and

wildlife and provides for recreation in and on the water be achieved by July 1, 1983.

(3) It is the national policy that the discharge of toxic pollutants in toxic amounts be prohibited.

(4) It is the national policy that Federal financial assistance be provided to construct publicly owned waste treatment works.

(5) It is the national policy that areawide waste treatment management planning processes be developed and implemented to assure adequate control of sources of pollutants in each State.

(6) It is the national policy that a major research and demonstration effort be made to develop technology necessary to eliminate the discharge of pollutants into the navigable waters, waters of contiguous zone, and the oceans.

It can be deeply recognized for us that the public in the U.S.A. has become more aware of water pollution and the climate of public opinion has remarkably changed to support the clean-up effort in the activities of Federal, State and municipal levels. Of course, without the public's support, it will be impossible to do the job of water pollution control. The passage of the 1972 Act is very significant that the goals are indicated clearly for the nation's commitment to clear water and the Act is certainly a commendable effort on the part of the Federal Government to upgrade and/or preserve the chemical, physical and biological integrity of the nation's waters, and the public is encouraged by the Federal Government's recognition of the problems of water pollution and its determination to deal with these problems.

It is understandable in general that the concept of setting water quality standards for the nation's waters and the concept of specifying minimum treatment levels regardless of stream requirements are sound and agreeable. The continuing planning process mandated by Section 303 (e) of the Act is a also sound and necessary to take comprehensive countermeasure for pollution prevention. This kind of planning can lead to take a more effective view at the overall water quality needs in each river basin and also lead to keep a much more coordinated control program than has been the case. It seems that State Governments and municipalities would express their attitude to keep close coordination with federal government to implement the 1972 Act.

Under these circumstances it seems that the goals and policy of the Act should not be altered and all governments in each level and the public would try to attain the goals, even if there must be some change in the schedules to attain the goals.

3. MUNICIPAL SECONDARY TREATMENT

Section 301 (b) (1) (c) of the Act says in effect that by July 1, 1977 publicly owned treatment works shall produce either a secondary treated effluent, or an effluent subject to advanced waste treatment if this is required to meet stream standards. It seems to me that the requirement that all municipalities provided a minimum of secondary treatment by 1977 may be difficult in some instances because of the time needed to plan, design, and construct facilities and also shortage of budgetary action.

According to the Secondary Treatment Information on secondary treatment pursuant to Section 304 (d) (1) of the Act, secondary treatment may not be capable of meeting the percentage removal requirements during wet weather in treatment works which receive flows from combined sewers. For such works decision should be made on case by case basis as to whether any attainable percentage removal level should be. In any case it will need huge amount of fund to meet requirements in these cases. Section 202 (a) provides that the amount of any Federal grant for publicly owned waste treatment plant made under the Act shall be 75 per centum of the cost of construction. But actually the level of funding as evidenced by past experiences would fall far short of the need. On the contrary national pollutant discharge elimination system as provided with Section 402 of the Act will require to apply and insure compliance with Section 301, 302, 306, 307 and 403. Moreover in the NPDE system discharge from storm overflow on combined sewers to public on case by case basis.

When the permit program is coupled with the grant program for publicly owned treatment works as established under Title II of the Act, a real dilemma does result. Actually it would appear to us that the federal government is telling state and local municipality on the one hand through permit system to construct rapidly the needed publicly owned treatment works; and on the other hand telling same state and municipality through the grant program that they are entitled to receive 75% federal funding, but it will not be forthcoming in many cases in time to meet the 1977 deadline. These two hands make confusing. It is extremely necessary to resolve this problem so that federal government can proceed harmoniously to construct the necessary works and also to maintain and operate the works satisfactorily. The congress is always hurry to take action and determine the deadline in disregard of possible materialization, but the engineers should consider the attainable and reasonable deadline. What date is it reasonable to attain the goal of 1977? There must be many difficulties to cope with the time limitation of the Act for small communities. Judging from my knowledge I am still obliged to acknowledge that it will be too short to attain not only the deadline of 1977 but also the ambitious national goals. It is easy to propose ammendments that will establish reasonable compliance schedules of the Act. But in the other hand we should pay attention that frequent ammendments might lose confidence for the national policy in the public.

We have same problems in Japan. In our Japanese practice when the environmental quality standard is fixed in some river basins, it will be usual to determine the deadline to attain the standard in each river basin according to the classification of environmental quality standard, and the deadline itself will not be so rigid, for instance, it will be indicated in such a way that 'within five years or as soon as possible within ten years' either by industrial waste treatment or publicly owned treatment plant. But it is still in many cases impossible to keep the deadline because of various difficulties such as shortage of budget available, lack of well-trained engineers etc. In addition to this in most of Japanese large cities the combined sewer system has been taken and in many cases storm-overflows do exist on sewers.

In these cases it needs huge amount of money to meet the requirements which will be decided on the case by case basis.

4. FINANCING

In Japan a significant problem related to the water program involves acute future financing of municipal waste treatment works. It seems to me that the situation on this matter in U.S. will be same as in Japan. Expanded eligibilities, more stringent abatement requirements, and increased construction costs have combined to raise the demand for federal grants far beyond the levels contemplated when the 1972 Act was enacted.

EPA's 1973 needs survey conducted under Section 516 (b) (2) of the Act identified a potential demand of 45 billion dollar from the Federal Government to fund project estimated to cost 60 billion dollar. A recent needs survey of 1974 shows that the cost of construction of all needed publicly owned treatment works in all of the States is 115 billion dollar. But when the costs for treatment and/or control of storm waters are included, the total costs will come to 350 billion which will be a huge sum of money to take a budgetary measure.

It seems that U.S. EPA is looking at such alternative financing strategies as differences in funding, in matching formulas, and in eligibility requirement to meet the deadline. I would like to follow such a financing strategies in the U.S. EPA's policy, because in Japan the Third Five-Year Plan (1971 ~ 1975) for Sewage Works will be completed by being invested amounting 2,600 billion yen, but it is expected to have another Fourth Five-Year Plan (1976 ~ 1980) for Sewage Works carried out.

The Ministry of Construction has made it clear that the total cost estimates for construction of publicly owned wastewater treatment plants and collecting sewers will need another 16,000 billion yen to attain the environmental quality standard in each river basin in Japan. Japanese Environmental Agency is responsible to fix environmental quality standard, and has already completed to fix it in about 300 river basins. We have continued arguments in relation to financing on sewage works and how to meet with needs and requirements.

Recently Ministry of Construction has requested to Ministry of Finance that the Fourth Five-Year Plan for Sewage Works should be appropriated 11,000 billion yen for the budget. Even if such program is proceeded completely the target dates will be postponed for some years, but Ministry of Finance has still showed unwillingness for such a huge budgetary request.

Under these circumstances we shall have to look for alternative financing strategies to attain environmental quality standard.

5. OCEAN OUTFALL AND OCEAN DUMPING

5.1 OCEAN OUTFALL

The U.S. EPA published information on secondary treatment under Section 304 (d) (1) of the 1972 Act. It described the minimum level of effluent quality attainable by secondary treatment in terms of the parameters BOD, SS, PH and B. Coli. It requires, in general, 85% removal or 30 mg/ℓ of BOD and SS whichever is more stringent. All publicly owned treatment works must provide treatment at least to this level by July 1977. We realize the 1972 Act and its legislative history clearly shows that the regulation is principally to be based on the capabilities of secondary treatment technology and not ambient water quality effects.

In the case of more stringent limitation, including those necessary to meet water quality standard required to implement any applicable water quality standard established pursuant to the 1972 Act, it shall be obliged to provide more advanced waste treatment technology. In the sense of necessity to meet water quality standard it will be necessary to take in account of capabilities of various kinds of treatment and decide the effluent limitation. Particularly it seems that biological oxygen demand is not a problem in ocean waters because there exists an abundant supply of oxygen available for degradation of municipal wastes.

Is there any possibility to reconsider the effluent limitations and their enforcement to take in account of water quality standard in each case particularly in the case of ocean outfall. For such treatment works the decision must be made on a case by case basis as to whether any attainable percentage removal level can be defined.

5.2 OCEAN DUMPING

We understand that it is the policy of the U.S. EPA to regulate the dumping of all types of materials into ocean waters and to prevent or to regulate strictly the dumping or other discharge into ocean waters of any material in quantities which would adversely affect human health, welfare, amenities, or the marine environment, ecological potentialities, or plankton, fish, shellfish, wildlife, shorelines or beaches.

It seems that the Section 403 (c) of the 1972 Act requires that application for permits for the dumping or other discharge of any materials into the marine environment be evaluated on the basis of the impact of the materials on the marine environment and marine ecosystems, on the present and potential uses of the ocean, and on the economic and social factors involved.

The regulation of ocean dumping may vary according to the types of waste materials as follows;

(1) The dumping of some types of waste materials into the marine environment is prohibited and will not be approved by U.S. EPA under any circumstances. Such prohibited waste materials are identified as follows;
   i) High-level radioactive wastes
   ii) Materials produced for radiological, chemical or biological warfare.
   iii) Materials insufficiently described in terms of their physical, chemical or biological properties to permit evaluation of their impact on marine ecosystems.

(2) The disposal of some types of waste materials into the marine environment is strictly regulated to prevent or minimize known or adverse effects on the aquatic ecosystem or human health and welfare. These materials and limiting concentrations and conditions upon the disposal of these materials are given as follows in the criteria; and these materials will be considered as trace contaminants when they are present in sewage sludge.

i) Organohalogen compounds
ii) Mercury and its compound: Solid state: Less than 0.75 mg/kg
Liquid state: Less than 1.5 mg/kg
iii) Cadmium and its compounds; Solid state: Less than 0.6 mg/kg
Liquid state: Less than 3.0 mg/kg
iv) Total oil
Do not produce visible surface sheen in an undistributed water sample when added at a rate of one part waste material to 100 parts of water.

(3) The disposal of some types of waste materials in subject to less strict regulation and permission because of the minimal adverse environmental effects to be anticipated.

i) Waste materials containing none of the materials mentioned above (1), (2) with limiting permissible concentration may be regarded as none-toxic in the marine environment.
ii) Solid waste
Solid waste of natural minerals or materials compatible with the marine environment may be generally approved for ocean disposal.
iii) Disposal of dredged materials

(4) Materials requiring special care are as follows;
Arsenic Lead Copper Zinc Selenium
Vanadium Beryllium Chromium Nickel
in these cases it is obliged that the applicant can demonstrate that the material proposed for disposal meets the limiting permissible concentration of total pollutants considering both the pollutants in the waste material itself and the total mixing zone available for initial dilution and dispersion.

Under these regulation for ocean dumping of waste materials I assume that in U.S.A. suggestion must be made so that the general permit could be used to allow the dumping of municipal sludge, because I am told that there were many experiences for dumping of municipal sludge into marine environment without any impact for the cited environment.

Ocean dumping in Japan is prohibited as a rule except for some cases when dumping is permitted by the regulation relating to the Marine Pollution Prevention Act 1970 and policy of Japanese Government on this matter is going in the same line as in U.S.A. At the moment ocean dumping of municipal sludge in Japan is almost impossible because of strong opposition from fishermen, but some members of the scientific community including fishery science are contended that biodegradable organic matter such as municipal sludge with some nutrients will not adversely effect on marine environment or plankton or fish. It is extremely necessary to conduct large-scale research demonstration in this field, but actually we have now in Japan strong opposition even for such experimental research works.

I am told that a permit for dumping of materials into the ocean as part of research into the impact of materials on the marine environment may be issued by U.S. EPA Administrator when he determines the scientific merit of the proposed project outweighs the potential damage that may occur from the dumping under the following conditions;

i) The applicant provides to the Administrator a detailed statement of the proposed project, including an assessment of the probable environmental impact carrying out the project.

ii) There is public notice and opportunity for public hearing.

iii) Research permit will be issued for no longer than 18 months, but may be renewed after review by the U.S. EPA Administrator.

The research permits of ocean dumping may be good idea to solve our problems in Japan demonstrating the actual examples. We shall be much obliged if you will show us some examples of research permits in which research works on ocean dumping of municipal sludge have been carried out.

6. PRETREATMENT

Pursuant to Section 307 (b) of the Federal Water Pollution Control Act Ammendments of 1972 the pretreatment standards for introduction of pollutants into publicly owned treatment works can be described in terms of the following pollutants;

(1) Pollutants which are determined not to be suscepti[illegible]

(2) Pollutants which would interfere with the operation and maintenance of such treatment works

This means that pretreatment standards are designed to achieve both to prevent the discharge of pollutants which pass through such works inadequately treated – incompatible – and to protect the operation of publicly owned treatment works – prohibited wastes –.

I realized that the definitions of compatible pollutant, incompatible pollutant, joint treatment works, and major contributing industry are very important to understand practical implementation of the 1972 Act. It should be noticed that compatible pollutant will be the pollutant identified in the National Pollutant Discharge Elimination System (NPDES) permit if the publicly owned treatment works does remove such pollutants to a substantial degree (about 80% removal) including COD, TOC, P, N, Fats, Oil and Grease. It should be noticed also that under the NPDES all point source including publicly owned treatment works to must obtain a permit for the discharge of wastewater to the navigable waters of the United States, but permits will not be required for industrial wastewater discharging into public sewers, and the effluent limitations for a pollutant in the discharge from a publicly owned treatment works will be individually determined by the regulating agency being based upon Secondary Treatment Information, Toxic Effluent Standards, Water Quality Standard whichever the most stringent limitation.

Under these circumstances it is very important to have strong and clear pretreatment policy considerations. Judging from the definition of incompatible pollutants it may be recognized that incompatible pollutants will be materials to inhibit biological activities and to give accute or chronic effects to aquatic life. I do agree that generally speaking the incompatible pollutants in amounts greater than would be permitted if the user discharged directly into navigable waters should be confined into the in-plant site, and accordingly the pretreatment standards for incompatible pollutants are the same as the standards for direct discharge into navigable waters. Pretreatment is required for incompatible pollutants to the levels of best practicable control technology currently available as defined for industry categories under Section 304 (b) of the 1972 Act.

Incompatible pollutants including toxic effluent will not be designed to protect sewer systems as a general principle, but to protect aquatic life in the navigable waters concerned from both acute and chronic impacts, but we have to pay attention that such incompatible pollutants as heavy metals may accumulate in the sludge generated by the publicly owned treatment works.

Recently we have been doing experimental works of sludge application on farm land, and in doing so we have realized that farmers are so nervous to apply sewage sludge on their farm land due to heavy metal accumulation in the soil. From these points of view it is recognized that we should review pretreatment standards and should keep such materials as heavy metals in the sludge to such a degree that sludge may be applied safely to farm land.

It seems that the Pretreatment Standard is deficient in that it fails to impose specific numerical limitations on the discharge of pollutants of prohibited waste and also incompatible pollutants. Of course it can be realized that the necessary degree of prohibited wastes and incompatible pollutants in each treatment works depends on the concentration of pollutant in the treatment works itself rather than the concentration of each user's effluent. It will, however, be quite required that a national pretreatment standards should be workable and enforceable and for that purpose it must be prescribe the permissible concentrations of particular pollutants of the user's effluents. Otherwise the user will not know what step he must take to comply with the pretreatment standards.

It is my understanding that joint treatment of industrial and municipal wastewaters in the same plant is generally a desired practice, because there are some advantages which are in the terms of increased flow which can result in reduced ratio of peak to average flows, savings in capital and operation and maintenance expenses due to the economics of large-scale treatment facilities and better use of manpower and land etc., and joint treatment of domestic sewage and adequately pretreated industrial wastewaters is encouraged where it is the economical choise including social costs.

Under stringent pollution control policy being taken in Japan, there is a tendency that municipalities will be unwilling to receive industrial wastewaters into public sewers due to difficult job relating to inspection and monitoring, but industries will be willing to discharge their wastewaters into public sewers.

The fact that an industry chooses to use a public sewer system rather than discharging his wastewater directly into the navigable waters should not involve getting away from penalty, and the industry concerned, of course, should provide his own pretreatment plant provided with currently best practicable control technology and also should pay the appropriate money to the municipality concerned according to pollutors pay principle, PPP.

7. USER CHARGE

Section 204 (b) (1) of the 1972 Act prohibits Federal grant assistance to a municipality unless it has adopted a proportionate user charge system to defray costs associated with operating and maintaining a facility including replacement. In the Federal guidline relating to user charges for operation and maintenance of publicly owned treatment works, it is written that Section (b) (1) of the Act provides that after March 1, 1973 Federal grant applicants shall be awarded grants applicants only after the Regional Administrator has determined that the applicant has adopted or will adopt a system of charges to assure that each recipient of waste treatment services will pay its proportionate share of the cost of operation and maintenance including replacement. The intent of the Act with respect to user charges is to distribute the cost of operation and maintenance of publicly owned treatment works to the pollutant source and to promote self sufficiency of treatment works with respect to operation and maintenance costs.

It seems that the problem is compounded because of an opinion of the Comptroller General which prohibits U.S. EPA from funding new projects in areas where municipalities are utilizing ad valorem tax systems to finance wastewater treatment operation and maintenance charges.

During the Third Conference there was a discussion presented by Metropolitan Sanitary District of Greater Chicago, and I quote: "Under our present system, we collect revenue from an ad valorem property tax together with an industrial surcharge when the discharge of an industry exceeds 3,650,000 gallons within a 12-month period. This method of funding has proven successful and the merits of changing this proven method are dubious. Another problem is presented in attempts to measure the discharge of each domestic user to achieve a fair allocation of the cost of treatment. The Metropolitan District of Greater Chicago plus 116 other municipalities. It is a practical impossibility to monitor the discharge of every domestic user in the Chicago metropolitan area. One suggested alternative is to correlate water consumption with discharge to the sewers and base the user charge on the volume of water consumption.

This might be feasible in many of the 116 municipalities outside the City of Chicago where water is metered but within Chicago there are about 350,000 nonmetered users. These users, mostly residential, are charged a flat rate for water. Under a direct user charge, about 350,000 water meters would have to be installed within the City of Chicago at a cost of approximately 60,000,000 dollars. The cost of reading these additional 350,000 meters is estimated to be somewhere around 3,000,000.00 dollars a year.

The contemplated user charge assumes that the sole function of a district is the collection and treatment of waste. The Metropolitan Sanitary District, however, is also charged by statute with the additional duties of flood control, maintenance of waterways, and the abatement of the pollution of waters from which any municipality might receive its water supply. This, of course, means the protection of Lake Michigan. These additional responsibilities would have to be funded by the ad valorem tax, so that even if a user charge is invoked, it would still be necessary to levy the ad valorem tax.

There would also exist the problem of enforcing the user charge if a user become delinquent. When you consider that the Metropolitan Sanitary District serves 5.5 million people and 11,000 industries in the metropolitan Chicago area, it is easy to see that we would be forced to spend a great deal of time in collecting or attempting to collect deliquent charges. Yet, this would be only a small part of the total cost of administration of user charges. The major expense would be in the routine billing and collection from literally hundreds of thousands of users. Now – in our present tax system, we succeed in collecting 89% of the tax levy. This money is forwarded to us by the county collector automatically and routinely without cost. The difference here speaks for itself.

How then the user charge benefit the taxpayer? I suppose it is possible that a system can be devised where the cost for each user will be more proportionate than it is now. Bear in mind, however, that a proportionate share under the user charge will probably mean a greater cost to the average homeowner than he is now paying in taxes, when you consider the administration and collection costs, and a collection experience that cannot be expected to approach 89%.

The Metropolitan Sanitary District is not taking a hostile position to the Federal requirements. We are simply advocating that our present method of financing by an ad valorem property tax coupled with an industrial surcharge meets the intent of the EPA standard for user charge."

It seems that the presentation by Metropolitan sanitary District of Greater Chicago does tell us whole story for user charge system in the case of cities with proud history of sewage works operation of their own. And it can not be helped to say that in these cases it would be reasonable to give such cities discretion utilize modified user charge system to pay their share of operation and maintenance costs.

The basic principle of user charge system in Japan which is suggested by Ministry of Construction will be described as follows;

(1) Construction cost of facilities for sewerage and sewage treatment shall be covered by public expenses including grant, public loan, local tax and so on.

(2) Operation and maintenance cost of facilities excluding cost of replacement shall be paid by the revenue from the user charge.

(3) User charge should be divided into two classes.
   i) Domestic user charge
   ii) Industrial user charge

   Domestic user charge shall cover only operation and maintenance cost excluding replacement of facilities and be decided by quantity discharged into sewer.

   Industrial user charge shall cover not only operation and maintenance cost but also cost of replacement by reservation of depreciation account. That portion should be recovered in accordance with so called Pollutors Pay Principle and be decided by quantity and quality requirement discharged into sewer.

(4) User charge system on a graduated scale shall be taken in order to economize water consumption and so quantity discount to large volume users will not be acceptable.

(5) Water consumption with discharge to the sewer should be correlated by water meter and water consumption by well from ground water with discharged to the sewer shall be estimated on the basis of pump capacity and its running hours.

Philosophy of these basic principle is: pollution should be stopped and controlled at the source. One of the most effective methods of reducing the pollution load into the sewer caused by industrial effluents is to make a user charge which is based on a sliding scale in accordance with their quantity and quality. In this way an incentive can be given to the trader to reduce his discharge of waste from his factory and in doing so to pay less user charge, by re-using water, by making minor modifications in manufacturing processes, by recovering by-products or by some means. Some remarkable results have been achieved in this way with profit to the trader, and with great advantage to sewage treatment operation, and with considerable resulting contribution to the national economy including water resources. Recently experiences of sewage works operation in Osaka City, Japan, have shown the remarkable reduction of industrial effluents into the public sewer by employing the suggested user charge in terms of industrial effluents. With regard to water resources investigation the Ministry of Construction published that by 1985 the 1970 output of water conservation works in Japan will be needed to be about 1.5 times as a nation-wide. The major water deficiency areas are in large metropolitan areas such as Tokyo, Osaka, Nagoya, Kobe, Fukuoka, Sendai etc., and even for the deficiency areas the high costs of building dam and transmission may link that the transfer of water from remote dam site has no obvious advantage from view points of costs.

The future water demand estimated by the Ministry of Construction is shown in the Table 7.1. Firm long-term forecast are not be available, but there seems to be obvious limit to the growth in demand for water from water undertakers, both to meet increased domestic consumption consequent upon rising social living standards and to serve the growing demands of industry. It has now been realized that the fast increasing consumption of water, both domestic and industrial, will make imperative a much greater reuse of water and also economical usage in both domestic and industrial purposes in terms of consumption in nearly all over Japan.

Under these circumstances it appears that the user charge system on a graduated scale will be more effective in such a way that the users including domestic and industrial consumers may try to save water consumption to pay less user charge, and in doing so an incentive can be given to dischargers also.

Each house is provided with its own water meter in almost every municipality in Japan and water consumption with sewage discharge into the sewer can be correlated. In this connection there is no difficulty to employ user charge system for us. Recently the charge system in terms of water and sewage has been revised in Tokyo Metropolitan Government as shown in Table 7.2. The basic principle of this revision is going in the same line as mentioned above.

In this cases of water consumption with discharge of same amount, say monthly consumption, 10 m^3, 20 m^3, 50 m^3, 100 m^3, 1,000 m^3, calculation of water and sewage charge in each case under Tokyo Metropolitan revision is shown in

Table 7.1 Future Water Demand Estimation

(0.1 billion cub. m. per annum)

Region	1970				1985			
	Domestic	Industrial	Agricultural	Total	Domestic	Industrial	Agricultural	Total
Hokkaido	3.6	9.6	49.7	62.9	8.6	32.4	56.9	97.9
Tohoku	9.1	16.1	129.5	154.7	17.5	46.5	143.8	207.8
Kanto	34.0	33.0	83.3	150.3	69.0	65.8	87.5	222.3
Hokuriku	2.0	10.2	28.4	40.6	4.6	20.6	28.9	54.1
Tokai	10.6	38.7	46.4	95.7	27.1	64.2	52.8	144.1
Kinki	21.2	28.4	44.5	94.1	39.7	46.3	42.8	128.8
Chugoku	5.5	16.1	48.8	70.4	12.6	30.8	52.2	95.6
Shikoku	2.6	9.0	23.1	34.7	6.2	22.8	26.4	55.4
Kyushu	7.1	13.5	69.9	90.5	21.3	41.4	94.2	156.9
Total	95.7	174.6	523.6	793.9	206.6	370.8	585.5	1,162.9

Table 7.2 List of Water Charge and Sewer User Charge in Tokyo Metropolitan Government

List of Water Charge (Yen)

Old			New		
Basic charge	Range (m^3)	Specific charge due to quantity	Basic charge	Range (m^3)	Specific charge due to quantity
100	0 ~ 8	0	300	0 ~ 10	0
100	9 ~ 18	20	300	11 ~ 20	60
120	19 ~ 30	25	400	21 ~ 30	75
120	31 ~ 50	28	400	31 ~ 100	90
140	51 ~ 100	45			
500	101 ~ 200	55	500	101 ~ 200	120
500	201 ~ 1,000	68	2,400	201 ~ 1,000	150
12,000	1,000 ~ over	75	17,000	1,000 ~ over	180

List of Water Charge (Yen)

Old			New		
Basic charge	Range (m³)	Specific charge due to quantity	Basic charge	Range (m³)	Specific charge due to quantity
80	0 ~ 8	0	100	0 ~ 10	0
	9 over	10	100	11 ~ 20	20
			300	21 ~ 30	30
			1,200	51 ~ 100	35
			2,950	101 ~ 200	45
			7,450	201 ~ 500	55
			23,900	501 ~ 1,000	55
			56,450	1,000 over	75

Table 7.3 Water and Sewage Charge as shown in Yen/cub. m. in Tokyo

	Water Charge		Sewer User Charge	
	Old	New	Old	New
10 m³	14.00	30.00	10.00	10.00
20 m³	18.50	45.00	10.00	15.00
50 m³	23.60	71.00	10.00	28.00
100 m³	34.50	80.50	10.00	40.50
1,000 m³	63.71	142.05	10.00	80.25

Table 7.3.

It seems to me that the sewer user charge system can be expanded to apply to pollution charge system in overall water pollution control. Levying charges on those who discharge wastewater into public water body in proportion to the quantity and the nature of the wastewater. By and large the charge mechanism provides just such an incentive. Instead of having water as a natural resource of infinite capacity, we find ourselves in a position where water is a limited resource which urgently needs to be managed. The preparedness of industries to reduce their waste effluents was strongly stimulated by the fact that the annual charge levied for financing water quality management is directly related to the volume and the nature of the effluent. This made it economically attractive for any industry to try and find a solution to the problem of how to limit the amount of waste to be discharged.

Pollution should be reduced in the point where the costs of doing so are covered by the benefits from the reduction of pollution. What is required, ideally, is some incentive to pollutors to reduce pollution up to the point where the costs to them of further pollution abatement would be greater than damage done by the pollution. This is why the ideal means of avoiding excessive pollution at least in principle, is to make the pollution pay a charge corresponding to the damage done by his pollution

This principle is attractive and justified, not only does it associate the cause with the consequence, but it has also become clear from practice that having to pay a charge makes an industry much more pollution-minded. This circumstance, combined with a strong permit policy, may lead to a considerable reduction of the pollution and may often change a potential pollutor into a supporter of water quality management.

Management of water resources should take into account social, economic, scientific and technical aspects and the sewer user charge system would be a key to water management by expanding it to pollution charge system.

8. INDUSTRIAL COST RECOVERY

Pursuant to Section 204 (b) (3) of the 1972 Act, "The grantee shall retain an amount of the revenues derived from the payment of costs by industrial users of waste treatment services, to the extent costs are attributable to the Federal share of eligible project costs provided pursuant to this title as determined by the Administrator, equal to (A) the amount of the non-Federal cost of such project paid by the grantee plus (B) the amount, determined in accordance with regulations promulgated by the Administrator, necessary for future expansion and reconstruction of the project, except that such retained amount shall not exceed 50% of such revenues from such project. All Revenues from such project not retained by the grantee shall be deposited by the Administrator in the treasury as miscellaneous receipts. That portion of the revenues retained by the grantee attributable to clause (B) of the first sentence of this paragraph together with any interest thereon shall be used solely for the purpose of future expansion and reconstruction of the project."

It seems that a grantee must elect to compute industrial cost recovery amounts annually on projects completed within an established accounting period. Such fund also must be segregated in special accounts and accounting procedures must be established to permit audit at any time during the recovery period.

It will be possible to determine what percentage of the capacity of the sewage treatment plant is attributable to any industry, but if there are many major contributing industries which discharge their wastewater into sewage treatment plant, such a determination can be an immense even if not an impossible task. In these cases cataloging of industries for the purpose of cost proportioning would require the sewage works authority concerned to keep record on great amount, and the authority would have to know when there is a change in ownership, when one goes out of business, or when a new one comes, when one changes its manufacturing processes or raw materials to be treated in its factory, continuously revising each business' contribution.

I would like to have informations on this matter and good examples in practice which has been carried out satisfactorily with the requirements of Section 204 (b) (3) of 1972 Act.

Fourth US/JAPAN Conference
on
Sewage Treatment Technology
Paper No. 7

WET WEATHER FLOW AND COMBINED SEWER OVERFLOW ABATEMENT TECHNOLOGY

October 29, 1975
Washington, D. C.

Ministry of Construction
Japanese Government

WET WEATHER FLOW AND COMBINED SEWER OVERFLOW ABATEMENT TECHNOLOGY

K. Saito and M. Kashiwaya, PWRI, Ministry of Construction

1. Sewer Served Area and Status Qua in the Sewer Collection System in Japan . . 395
2. Investigation on the Storm and Combined Sewer Overflow in Japan 396
3. Quality of Combined Sewage and Urban Storm Water 398
4. A Proposal of Storage Tank 399

WET WEATHER FLOW AND COMBINED SEWER OVERFLOW ABATEMENT TECHNOLOGY

1. SEWER SERVED AREA AND STATUS QUO IN THE SEWER COLLECTION SYSTEM IN JAPAN

In 1974, 426 municipalities were undertaking sewage works in Japan. In 1975, 60 municipalities are launching sewage works.

In 1974, an area of 2,080 km^2 was sewered, accounting for 25.5% of the urban area in Japan. The population served by sewers are estimated to be 38 million.

In 1972, Ministry of Construction made a survey on the sewer collection systems. 316 municipalities responded to our inquiries. The results of the survey are shown in Table 1. It is found that 104 municipalities employed the combined sewer systems, while 96 cities used the separate sewer systems. There were 81 cities where the sewer systems were mainly of the combined type and partly of the separate type. Most of these cities applied the combined sewer systems to the urban area and the separate type to the suburbs.

In 35 cities, major portion of their service area had separate sewers with the remaining portion served by combined sewers. This type of sewer systems are classified into two; one in which separate sewers are used in steep areas, while combined sewers in plane areas, and another in which the newly developed residential zones in the suburbs use separate sewer systems while the most old urbanized areas are remaining unsewered. In the areas covered by the combined sewer systems, there live 26.31 million or 70% of sewer served population.

Table 1 shows a comparison between Japan and the United States as to the combined sewer systems.

In Japan, the cities employing the separate sewer systems have been increasing as combined sewer systems was criticized for its combined sewer overflow problems. Ministry of Construction also has been recommending the separate sewer systems for the purpose of water pollution control.

If the unsewered cities will be arranged with separate sewer systems, Japan's sewer systems will reach almost the same level as in the United States.

2. INVESTIGATION ON THE STORM AND COMBINED SEWER OVERFLOW IN JAPAN

Since 1967, Public Works Research Institute, Ministry of Construction has made investigations on the storm and combined sewer overflows.

The data concerning rainfalls, storm water and water quality for the 1867~69 period were collected from five cities ~ Tokyo, Nagoya, Toyohashi, Kyoto and Gifu. The data collected were aimed at the preparation of design manual for new sewerage plans and the evaluation of the existing combined sewer systems. The areas investigated were each different in population and land use from others.

The survey also included the examination of applicability of the rational method, measurement of inlet time and time of concentration, investigation of runoff coefficient, actual quality of sewage in the wet-weather conditions, and change in the rainfall runoff attendant on urbanization.

Although the surveys were completed in 1969, from 1970 till now the observation of rainfalls and rainfall runoff has been carried out in the catchment area of the Yabatagawa, Tokyo. In the guidelines for the sewerage which were established in 1972 by Japan Sewage Works Association, it is recommended that the unsewered areas should be served by separate sewer systems, because combined sewer system is liable to cause storm water overflow problems leading to water pollution. But, the guideline leaves how to improve the existing combined sewer system over for the future study.

For this reason, a survey on the improvement of the combined sewer system or its alternatives has been conducted since 1973. So far, the survey has tackled the problems incidental to the overflow from the combined sewer, flow rate, water quality and characteristics of combined sewage and storm water discharge, assessment of alternatives proposed, and preparation of a conceptual plan for a storm water storage tank with the Yabatagawa catchment (Tokyo) taken as a model.

In 1975, engineers from Ministry of Construction, local governments and Japan Sewage Works Agency rallied to establish "Committee on Combined Sewer Overflow Problems." The purposes of the Committee are to let its members investigate the sewer systems in their respective cities, to have them exchange the informations obtained with each other, and thereupon to study how to improve the combined sewer systems in the respective cities. The Central Government is in a position to appraise the achievements of each city, while the Committee, which appripriates the outlays for the collecting of data necessary for the Central Government to persue the problems, is going to undertake research activities from 1975 till 1980.

The results of the investigation will be used not only as data for the Government's long-term projection, but also for the planning of each city's combined sewer improvement programs and preparation of the design manual for sewage system.

On the other hand, the Tokyo Metropolitan Government has made various investigations along the Yabatagawa and the Momozonogawa. The themes for the investigations include the examination of infiltration loss in the urban area, applicability of RRL method, development of modified RRL method, examination of the storm water drainage system, preparation of pollutant discharge model, etc.

In 1975, Tokyo Metropolitan Government relocated the observation station to Ogu District for continued survey. The District is a plain, and storm water is drained by pumping. The sewerage always carries a large amount of stagnant water. Hence, the results of the survey may become different from those in the Yatabegawa and the Momozonogawa.

Osaka Municipal Government planned a combined sewage (storm water) sedimentation tank on the occasion of the repairs of the Nakanoshima Pump Station located at the center of the city. The tank will be composed of six basins each measuring 3.5 m in width, 20.2 m in length and 5.2 m in height. Two out of six were completed in 1975. Osaka Municipal Government will use these tanks for the measurement of the volume and quality of influent and examination of the feasibility of the similar tanks which will be planned and constructed from now on.

The results of survey, such as conducted by local governments will be also submitted to the Committee on Combined Sewer Overflow Problems.

Water pollution problems due to storm water discharge in the urbanized area is not solely developed by combined sewer overflow.

In the old cities, dry-weather flow has been increased with the progress of urbanization, which has resulted in a shortage of sewer capacity. Namely, in dry weather conditions, raw sewage is liable to be delivered to the receiving waters without treatment or even when the rainfalls are not so severe, overflows have happened.

Consideration to the alternatives to solve the combined sewer overflow problems in these cities should also cover the measures to make up the shortage of sewer capacity. Sewer separation will become one of effective measures, except in those heavily build-up areas which have high population density. In most of oldest cities, the conversion of the existing combined sewer systems into the separate one is almost impossible as various restrictions are present, and alternatives may have to be taken up. There are many alternatives proposed, and their practicability should be examined from various aspects.

The results of this kind of surveys conducted by U.S. EPA are very informative to us.

In the separate sewer systems on the other hand, urban storm water itself presents a serious pollution problem. Also, excessive infiltration into sanitary sewer systems is concerned about since it results in overloading to sewage treatment facilities. In the existing separate sewer systems, storm water flow mainly in gutters, natural water courses, etc. If the urbanization advances in the future, storm water runoff will overrun the capacity of these facilities. It will be therefore necessary to drastically reconstruct the storm drainage facilities, install a basin in the city in order to pool storm water temporarily. In some case porous pavement and the like which decrease the rainfall runoff coefficient should be incorporated.

3. QUALITY OF COMBINED SEWAGE AND URBAN STORM WATER

Public Works Research Institute has investigated the quality of combined sewage in Tokyo, Nagoya and Kyoto.

Table 2 shows test locations. The results of surveys are shown in Table 3 ~ 5.

According to a U.S. EPA survey report* the quality of combined sewer overflows are that of raw sewage or its half. In Japan, the tendency is almost the same. It is interesting to note that the quality is nearly the same for all that the way of living and rainfall conditions are quite different between the two countries (Table 6).

In the Yabatagawa District in Tokyo, the "first flush" is found noticeable at a rainfall of 10 mm/hr to 20 mm/hr, particularly when heavy downpours are seen in the beginning (Fig. 1). At 3 to 5 mm/hr of rainfall, it is not found precisely (Fig. 2).

In the case of high-intensity rainfall, dilution phenomenon is noticed (Fig. 3).

If two consecutive rainfalls are present, "first flush" occurs in second railfall, even when their interval is short especially the second one is heavy (Fig. 3, Fig. 4). What is noticeable in the first flush includes BOD, SS, etc., and nutrients, total-coli. are rarely seen.

In 1975, surveys on combined sewage have being conducted in 11 largest cities; Sapporo, Tokyo, Kawasaki, Yokohama, Toyohashi, Nagoya, Kyoto, Osaka, Hiroshima, Kitakyushyu, Fukuoka. As regards the storm water discharge into the separate sewer system, surveys were carried out in Gifu, Kobe and Yamagata for the purpose of contrasting them with the combined sewage. Table 7 shows the test locations and the results in Table 8 ~ 10.

Storm water discharges were found seriously polluted, and ocassionally these quality were similar to raw sewage. Such Heavy metals as Zn, Pb, were also included considerably in storm water discharge (Fig. 5). The mean value was poorer than the secondary effluent, showing little difference from the survey results in U.S.A. (Table 11). In Japan, gutters and natural small water courses have been used for the purpose of drainage where the separate sewer systems is applied. In dry weather conditions, pollutants deposite in them and are scoured away with rain water. This has been the major cause of the pollution of storm water from the separate sewer system.

In 1975, the survey on urban storm water has been continued in Kobe.

The total rainfalls in Japan are 1,500 mm/year to 2,000 mm/year, and typhoons visit, bringing about heavy downpours. But generally, Japan's largest cities have a high population density and use large quantities of water. Accordingly, the annual BOD and SS loading of the effluent from the sewage treatment plant are much more than those in the combined sewer overflow which is directly discharge into the receiving waters.

Namely, the effluent from the sewage treatment plant is always imposing higher loading on the receiving waters. Table 12 is the results of an estimate with a down town in Tokyo taken as a model.

It is clear from Table 12 that BOD loading of the combined sewer overflow accounts for no more than 20%.

---

* F.J. Condon "Storm and Combined Sewer Abatement Technology in the U.S.A. – An Overview –" EPA, Feb., 1974.

Most of nutrients are contained in sewage treatment plant effluents.

4. A PROPOSAL OF STORAGE TANK

There are a lot of proposals for overcoming the combined sewer overflow problems.

One such example is a storage tank. It is believed to have the following advantages.

1) The structure is simple and easy to operate.
2) The conventional sewage treatment technology is applicable.
3) Stage-wise construction is possible, and takes effect soon as local modification of sewage treatment facilities will do for the purpose.
4) Capacity of the existing sewer can be increased.
5) Wide fluctuation of storm water flow can be controlled and equalized.

Particular emphasis may be placed on the last merit since Japan often is hit by heavy downpours.

Pulbic Works Research Institute estimated the effects of the combined sewage storage tank with the Yabatagawa District, Tokyo as a model.

The District lies northwest of Tokyo, and covers an area of 541 ha with the combined dewer systems. The result of estimation are as follows; in this district, the calculated BOD and SS loadings are as shown in Fig. 6 namely, the removal rate of BOD and SS loadings consequent upon the discharge of storm water is only 22% and 28%, respectively.

By installing three tanks, 27,000 m^3, 54,000 m^3 and 81,000 m^3, 40% to 70% of the annual storm water discharge of 440 mm can be storaged. The storaged storm water is sent to the sewage treatment facilities in dry weather and then discharged into the receiving waters after secondary treatment. The results of this study indicates, the removals of BOD and SS loadings are from 60 to 70% and 70 to 80% respectively as shown in Fig. 7 Another benefit is the reduction of annual overflow frequency which is reduced to 42, 16 and 11 as against the theretofore 78.

Even if the district were served by separate sewer system, it will still deliver pollutants at an annual rate of 118 tons in BOD and 391 tons in SS as the storm water discharge is polluted.

The amount is tantamount to the installation of a storage tank with a capacity of 58,000 m^3.

Unfortunately, however, it is hardly possible to find out an open space allowing the construction of such a large facility.

It is therefore concluded that the storage tank should be constructed beneath public facilities such as park, parking lot, open space necessary at the time of earthquake etc. In this way, precious space can be used effectively.

Table 1 Relative Use of Combined Sewers

	Japan		U.S.A.	
Municipalities		%		%
Combined sewer	104	33	1,329	10
Combined and partialy separate	81	26		
Separate and partialy combined	35	11		
Separate sewer	96	30	12,000	90
Total	316			
Area (km^2)				
Combined sewer	1,526	73		
Separate sewer	552	27		
Total	2,078			
Population				
Combined sewer	2,631	69	36,236	29
Separate sewer	1,166	31	89,534	71
Total	3,797	31	125,770	

Table 2 Test Location (Combined Sewer)

City	Test location	Drainage area (ha)	Population	Land use
Tokyo	Yabatagawa	540.6	120,000	Residential, semi-industry
Kyoto	Chubu-daiichi	67.8	17,000	Residential
Nagoya	Tamitsu	51.4	9,900	Residential
Nagoya	Chitose-nambu	81.2	12,300	Semi-industry, harbar commercial

Table 3 Characteristics of Combined Sewage (1) (Yabatagawa, Tokyo)

Unit: mg/ℓ

	July 31, 1972				September 9, 1972				October 6, 1972				July 21, 1973			
	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.
BOD	61	353.0	94.2	38.5	56	158.0	104.6	69.4	44	240.0	102.3	30.0	35	317.4	121.7	62.3
S-BOD	30	63.0	29.0	15.1	28	55.8	37.1	25.6	22	66.1	33.6	18.5	11	94.3	65.3	49.8
T-S	30	886.0	569.0	305.0	28	535.0	462.9	324.0	22	1,076.0	531.0	353.0	11	1,025.0	548.1	294.0
SS	61	686.0	340.5	94.0	56	214.0	140.9	55.0	44	716.0	251.3	92.0	35	788.0	154.9	32.0
VTS	30	274.0	142.7	51.0	28	101.0	66.2	42.0	22	262.0	136.7	42.0	11	463.0	233.3	124.0
T-N*	16	17.3	11.3	7.3	15	14.7	14.2	13.2	11	33.8	14.6	10.0	6	36.1	24.7	14.4
T-P	16	2.88	1.79	1.07	15	2.78	2.51	2.15	11	5.92	2.78	1.74	6	7.20	4.56	2.50

	July 29, 1973				August 1, 1973				August 4, 1973				August 10, 1973			
	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.
BOD	70	147.0	89.0	31.3	60	246.3	88.6	16.6	83	177.7	70.1	19.8	35	244.5	99.5	19.6
S-BOD	34	448	18.3	10.2	24	73.8	19.4	10.8	28	63.4	8.8	10.2	11	88.2	36.7	23.5
T-S	19	475.0	196.9	185.0	24	1,133.0	341.4	131.0	30	680.0	142.7	134.0	11	1,092.0	323.5	197.0
SS	70	375.0	86.6	11.0	60	1,020.0	221.2	16.0	83	475.0	113.8	14.0	35	857.0	182.4	21.0
VTS	19	318.0	113.3	56.0	24	604.0	171.3	58.0	30	318.0	64.4	59.0	11	641.0	174.3	93.0
T-N*	10	29.4	9.8	10.1	12	11.7	4.3	3.7	–	–	–	–	–	–	–	–
T-P	10	3.10	1.17	0.80	12	1.90	0.54	0.50	–	–	–	–	–	–	–	–

	August 24, 1973				August 25, 1973				November 10, 1973				Summary	
	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Mean	Range
BOD	44	613.4	93.0	25.4	26	126.8	101.1	63.2	121	367.5	45.5	11.6	91.8	11.6 ~ 613.4
S-BOD	13	238.4	21.0	8.6	9	52.5	10.5	13.7	121	37.9	9.3	3.9	35.3	3.9 ~ 238.4
T-S	13	1,935.0	192.1	139.0	9	388.0	105.8	185.0	121	819.0	249.4	93.0	257.5	93.0 ~ 1,935.0
SS	44	1,768.0	152.2	39.0	26	243.0	118.0	64.0	121	680.0	162.9	14.0	175.0	11.0 ~ 1,768.0
VTS	7	984.0	419.5	104.0	9	234.0	183.9	88.0	121	378.0	89.7	31.0	163.2	31.0 ~ 984.0
T-N*	–	–	–	–	–	–	–	–	61	26.7	6.8	2.4	12.2	1.9 ~ 36.1
T-P	–	–	–	–	–	–	–	–	61	3.14	0.54	0.18	1.97	0.18 ~ 7.20

* Kjeldahl

Table 4 Characteristics of Combined Sewage (2)

	3 Aug., 67				22 Aug., 67				31 Aug., 67				Summary	
	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Mean	Range
BOD	29	1,176.0	88.1	55.3	23	108.8	49.35	22.0	27	219.5	71.37	19.2	69.0	19.2 ~ 1,176.0
SS	29	393.0	214.0	9.0	23	523.0	298.0	96.0	27	525.0	331.0	92.0	283.0	9.0 ~ 525.0
VSS	–	–	–	–	23	106.0	63.0	12.0	27	185.0	77.8	20.0	70.4	12.0 ~ 185.0

Table 5 Characteristics of Combined Sewage (3)
(Tamitsu & Chitose-nambu, Nagoya)

	19 July., 67				22 Aug., 67 (1)				22 Aug., 67 (2)				13 Sep., 67 (1)				13 Sep., 67 (2)				Summary	
	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Mean	Range
Tamitsu																						
BOD	40	296.6	72.6	4.2	8	59.7	84.7	5.0	14	80.6	63.9	3.5	19	51.3	44.0	5.7	12	16.8	55.9	7.1	64.2	3.5 ~ 296.6
SS	40	4,310.3	1,228.8	5.8	8	372.5	333.6	3.2	14	1,289.1	455.5	4.7	19	879.2	411.9	13.8	12	38.5	141.2	16.8	514.2	3.2 ~ 4,310.3
Chitose-nambu																						
BOD													21	70.0	32.3	7.3					32.3	7.3 ~ 70.0
SS													21	514.5	254.5	13.3					254.5	13.3 ~ 514.5

Table 6 Comparison of Quality (Strength) of Combined Wastewater Overflows

Type of wastewater and city	BOD_5 mg/ℓ Ave.	COD mg/ℓ Ave.	SS mg/ℓ Ave.	Total coliform MPN/100 mℓ
Raw Sanitary Flow	200	500	200	5×10^7
Primary Effluent	135	330	80	2×10^7
Secondary Effluent	25	55	15	1×10^3
Combined Sewer Overflows				
Atlanta, Ga.	100	–	–	1×10^7
Berkeley, Calif.	60	200	100	–
Brooklyn, N.Y.	180	–	1,051	–
Bucyrus, Ohio	120	400	470	1×10^7
Cincinnati, Ohio	200	250	1,200	–
Des Moines, Iowa	115	–	295	–
Detroit, Michigan	153	115	274	–
Kenosha, Wisconsin	129	464	458	2×10^6
Milwaukee, Wisconsin	55	177	244	–
Racine, Wisconsin	119	–	434	–
Sacramento, Calif.	165	238	125	5×10^6
San Francisco, Calif.	49	155	68	3×10^6
Washington, D.C.	71	382	622	3×10^6
Yabatagawa, Tokyo	92	–	175	–
Chubu-daiichi, Kyoto	70	–	283	–
Tamitsu, Nagoya	64	–	514	–
Chitose-nambu, Nagoya*	32	–	255	–

* 1 storm

Table 7 Test Location

City	Test location	Drainage area (ha)	Population	Land use
Gifu	Shimizugawa	106.4	24,500	Commercial
Kobe	Hanakuma	17.2	2,700	Residential, commercial, office
Yamagata	Midorimachi	13.7	1,000	Residential, commercial, office

Table 8 Quality of Urban Stormwater (1)
(Hanakuma, Kobe)

	3 Sep., 73				27 Oct., 73				21 Jan., 74			
	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.
BOD	24	980.0	99.4	13.0	51	136.0	31.0	9.5	52	743.0	104.6	15.0
S-BOD	14	57.6	13.2	8.5	27	43.0	11.0	2.7	27	70.0	11.9	5.0
SS	24	2,561.0	756.8	24.0	51	196.0	76.4	6.3	52	2,130.0	475.0	13.4
VSS	14	1,360.0	196.0	13.0	27	83.0	35.6	10.0	27	959.0	179.0	1.0
T-N*	14	65.8	15.8	2.3	27	10.7	3.8	1.4	27	71.0	12.3	1.5
T-P	14	1.15	0.73	0.36	27	2.06	0.67	0.38	27	2.96	0.68	0.32
Grease**	14	538.3	31.7	1.0	27	28.7	6.3	0.1	27	138.0	22.7	1.0
Total MPN	14	250,000		3,100	27	77,000		80	27	500,000		2

*: Kjehldahl **: Soxhlet method

	1 Sept., 74				1 Oct., 74				4 Feb., 75			
	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.
BOD	39	136	17.7	5.2	70	85.6	7.4	1.83	20	99.7	65.9	10.3
S-BOD	20	6.4	3.5	2.2	35	13.1	2.5	1.46	10	6.26	6.1	4.28
SS	39	417	97.8	27.2	70	381	78.6	9.4	20	630	504.0	24.5
VSS	20	86	22.0	5.6	35	111	17.7	2.8	10	190	156.3	12.0
T-N*	20	11.1	1.96	0.9	35	7.4	1.27	0.62	10	9.12	9.33	3.28
T-P	20	1.08	0.22	0.12	35	0.49	0.14	0.05	10	0.65	0.64	0.19
Greass**	20	12.4	3.8	1.6	35	9.7	2.40	0.2	10	41.8	35.6	4.0
Total MPN	20	61,000	–	400	35	35,000	–	1,300	10	17,000	–	250

*: Kiehldahl **: Soxhlet method

	6 Apr., 75				Summary	
	Sample No.	Max.	Mean	Min.	Mean	Range
BOD	27	142	57.0	11.3	54.7	980 ~ 9.5
S-BOD	14	17.9	10.9	5.9	8.4	70 ~ 2.2
SS	27	863	423.3	10.4	344.6	2,561 ~ 6.3
VSS	14	226	92.5	4.0	99.9	1,360 ~ 1.0
T-N	14	18.8	7.27	3.75	7.4	71 ~ 0.6
T-P	14	0.55	0.29	0.05	0.48	2.96 ~ 0.05
Grease	14	57.9	16.0	2.1	16.9	538 ~ 0.1
Total MPN	14	40,000	–	370	–	500,000 ~ 2

*: Kiehldahl **: Soxhlet method

Table 9 Quality of Urban Stormwater (2)
(Midorichyo, Yamagata)

	19 Sep., 74				15 Oct., 74				22 No., 74				14 Dec., 74				Summary	
	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Sample No.	Max.	Mean	Min.	Mean	Range
BOD	48	20.3	6.90	3.1	30	55.4	45.7	24.2	48	70.4	35.8	18.3	18	60.7	39.8	25.0	32.0	70.4 ~ 3.1
S-BOD	24	8.7	3.61	1.8	15	35.6	11.4	12.2	24	50.3	10.0	8.9	9	52.4	15.3	21.0	10.1	52.4 ~ 1.8
SS	48	103.0	37.6	8.3	30	131.0	79.6	59.5	48	175.7	93.9	27.7	18	279	246.7	222	114.5	279 ~ 8.3
VSS	24	47.5	19.0	3.6	15	76.5	23.4	39.0	24	116.7	33.3	28.2	9	20.7	8.72	11.9	21.1	117 ~ 11.9
T-N*	24	3.9	0.87	N.D	15	11.2	3.42	1.8	24	0.3	0.08	N.D	9	0.3	0.12	N.D	1.12	11.2 ~ N.D
T-P	24	0.6	0.18	0.09	15	1.67	0.77	0.42	24	0.57	0.19	0.06	9	0.29	0.18	0.12	0.33	1.67 ~ 0.06
Grease**	24	13.0	8.50	4.3	15	11.8	7.75	5.4	24	15.9	8.95	4.1	9	23.7	19.2	15.2	11.1	23.7 ~ 4.1
Total MPN	24	64,000	–	1,100	15	120,000	–	600	24	9,200	–	220	9	610	–	210	–	120,000 ~ 210

*: Kjehldahl **: Soxhlet method

Table 10 Quality of Urban Stormwater (3)
(Shimizugawa, Gifu)

	6 July., 67				12 July., 67 (1)				12 July., 67 (2)					
	Samples	Max.	Mean	Min.	Sample	Max.	Mean	Min.	Sample	Max.	Mean	Min.		
BOD (mg/ℓ)	40	14.3	11.6	3.83	82	13.[illegible]	7.3	3.2	65	11.3	7.3	4.11		
SS (mg/ℓ)	40	268.0	145.0	18.0	82	78.0	43.0	30.0	65	162.0	71.0	9.0		
VSS (mg/ℓ)	40	116.0	75.0	10.0	–	–	–	–	–	–	–	–		
	19 July., 67 (1)				19 July., 67 (2)				22 Aug., 67				Summary	
	Samples	Max.	Mean	Min.	Sample	Max.	Mean	Min.	Sample	Max.	Mean	Min.	Mean	Range
BOD (mg/ℓ)	44	14.2	8.7	4.03	34	10.7	6.8	5.04	85	13.9	7.2	3.43	8.15	3.2 ~ 14.3
SS (mg/ℓ)	44	476.0	251.8	46.0	34	88.0	45.2	13.4	85	208.0	113.0	57.5	111.5	9.0 ~ 476.0
VSS (mg/ℓ)	44	110.0	62.0	24.0	–	–	–	–	–	–	–	–	68.5	10.0 ~ 116.0

Table 11 Comparison of Quality (Strength) of Storm Water Discharges

City	BOD_5 Ave.	COD Ave.	SS Ave.	Total MPN/100 mℓ
Ann Arbor, Michigan	28	–	2,080	–
Des Moines, Iowa	36	–	505	–
Durham, N.C.	32	224	–	3×10^5
Los Angeles, Calif.	9.4	–	1,013	–
Madison, Wisconsin	–	–	81	–
New Orleans, La.	12	–	26	1×10^6
Sacramento, Calif.	106	58	71	8.10^5
Tulsa, Oklahoma	11	85	247	1×10^5
Washington, D.C.	19	335	1,697	6×10^5
Hanakuma, Kobe	55	–	345	
Midoricho, Yamagata	32	–	279	
Shimizugawa, Gifu	8	–	112	

Table 12 Estimated Annual Load of Pollutants from Urban Land

Pollutant	Secondary effluent		Primary effluent		Combined sewer overflow	
		%		%		%
Flow (10^3 m^3/year)	145,990	92.6	2,205	1.4	9,486	6.0
BOD (t/year)	2,845	73.1	267	6.9	777	20.0
SS (t/year)	3,712	67.1	306	5.5	1,518	27.4
TKN (t/year)	2,801	95.4	58	2.0	78	2.6

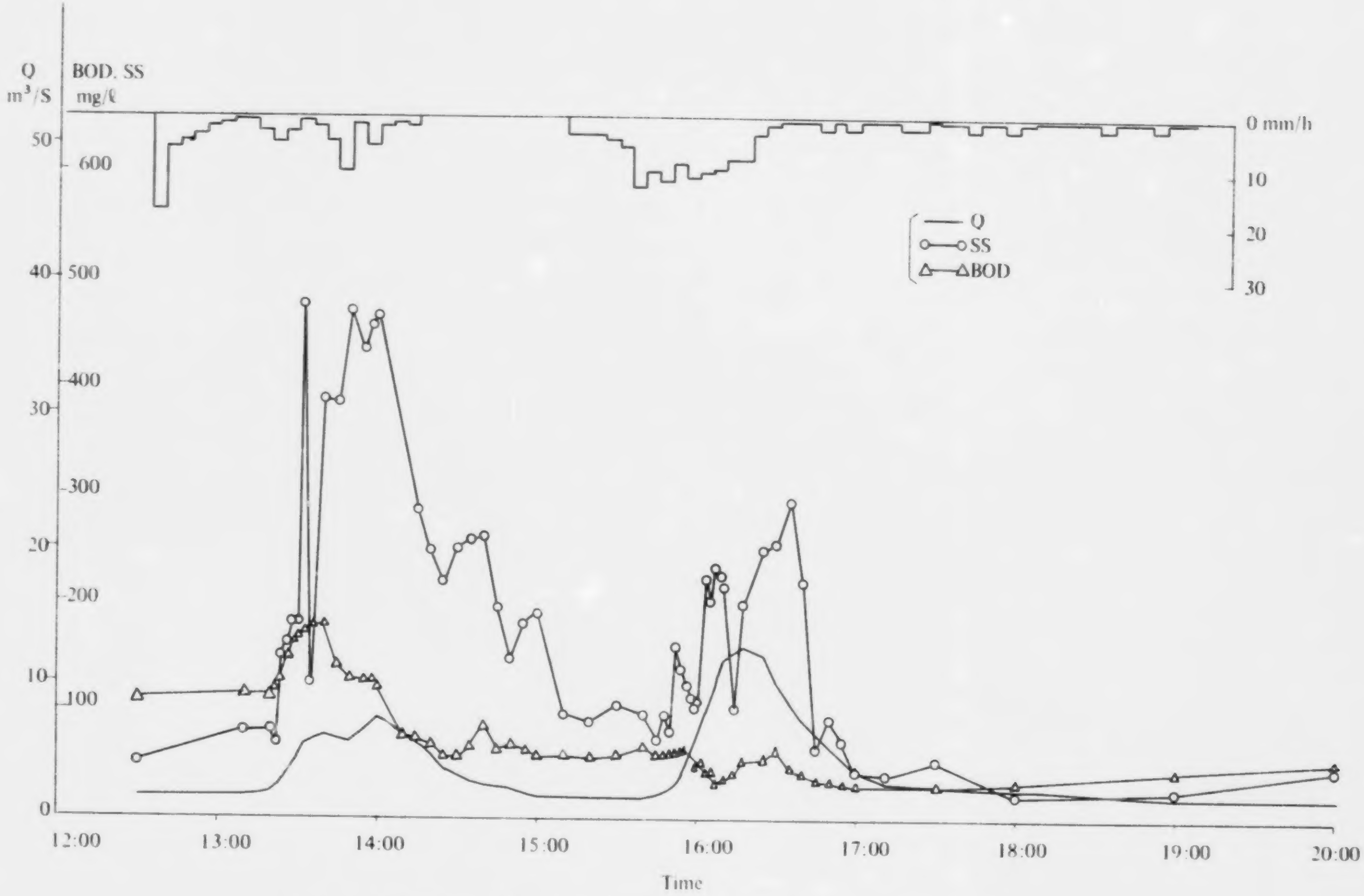


Fig. 1 Relationship BOD, SS to Rainfall (1 March 73, Yabatagawa, Tokyo)

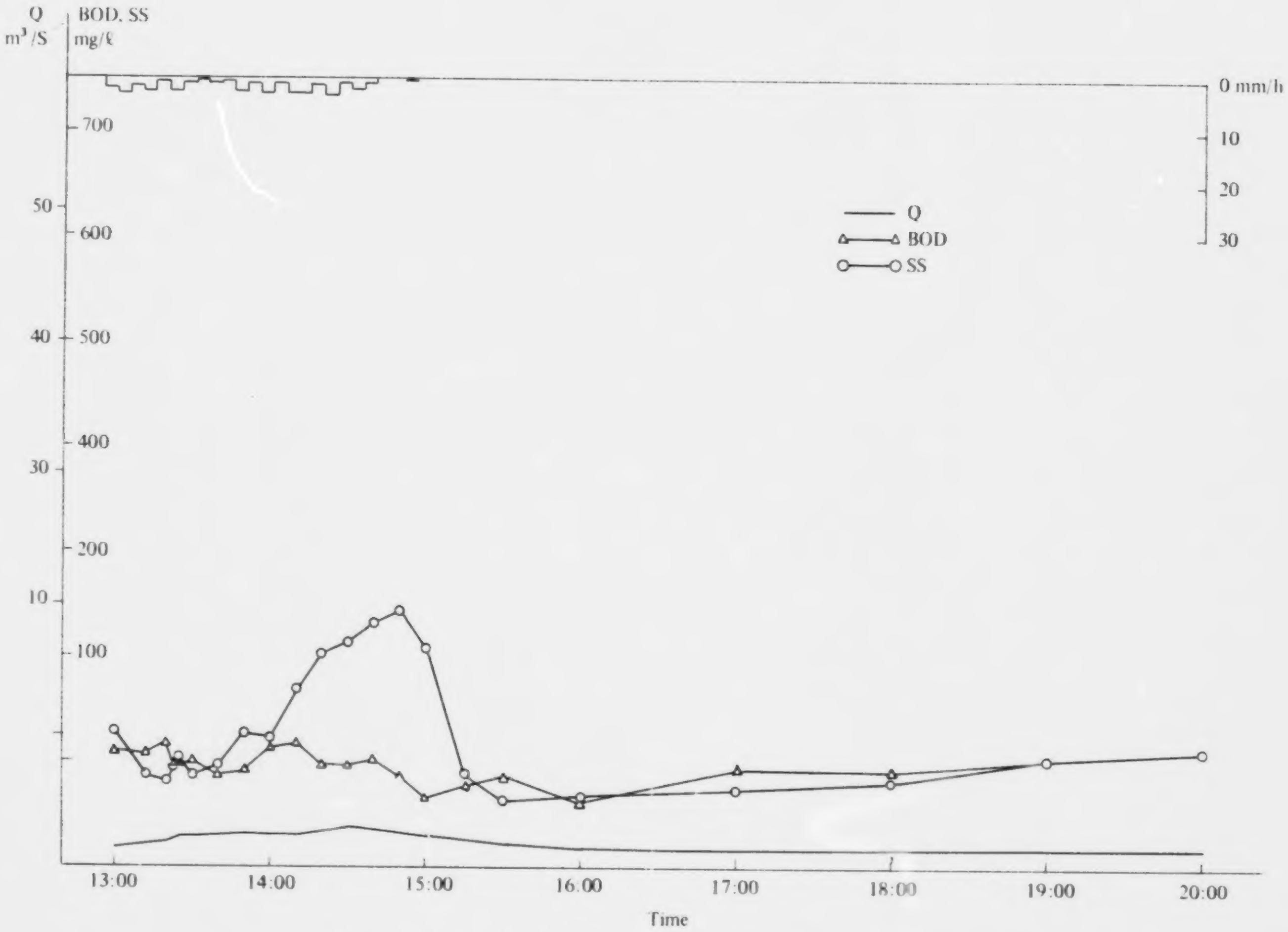


Fig. 2 Relationship BOD, SS to Rainfall (25 Aug. 73, Yabatagawa Tokyo)

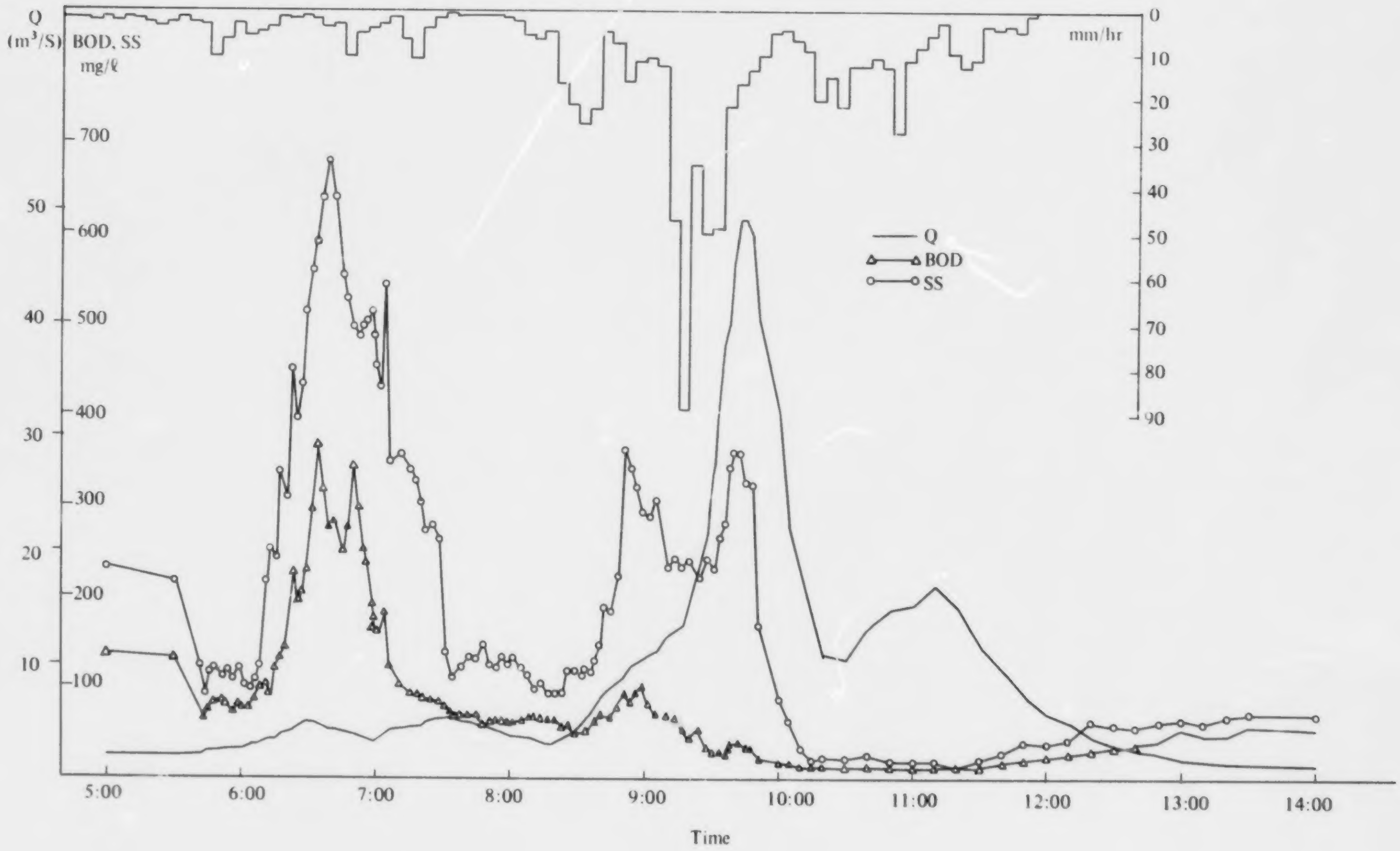


Fig. 3 Relationship BOD, SS to Rainfall (10 Nov. 73, Yabatagawa, Tokyo)

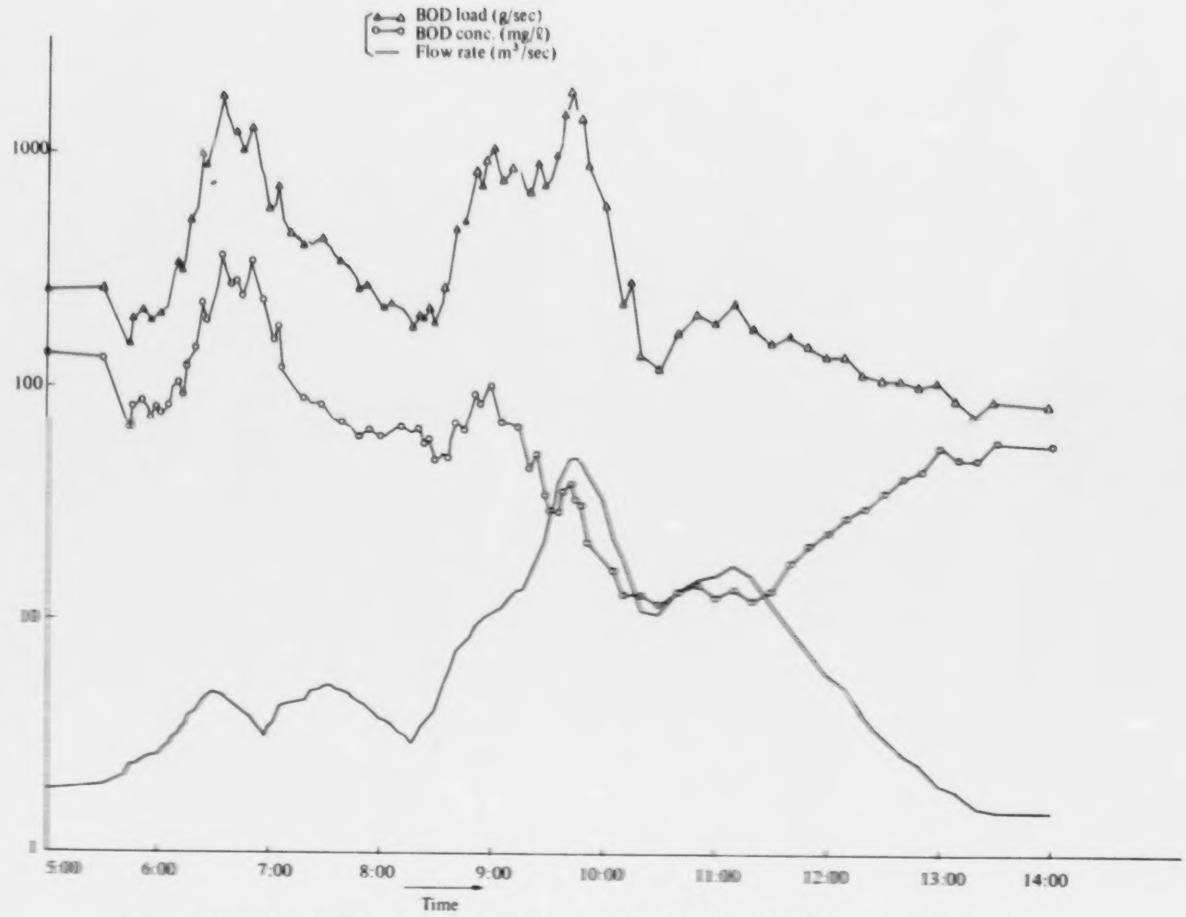


Fig. 4 Variation of BOD Load & BOD Concentration (10 Nov. 73, Yabatagawa, Tokyo)

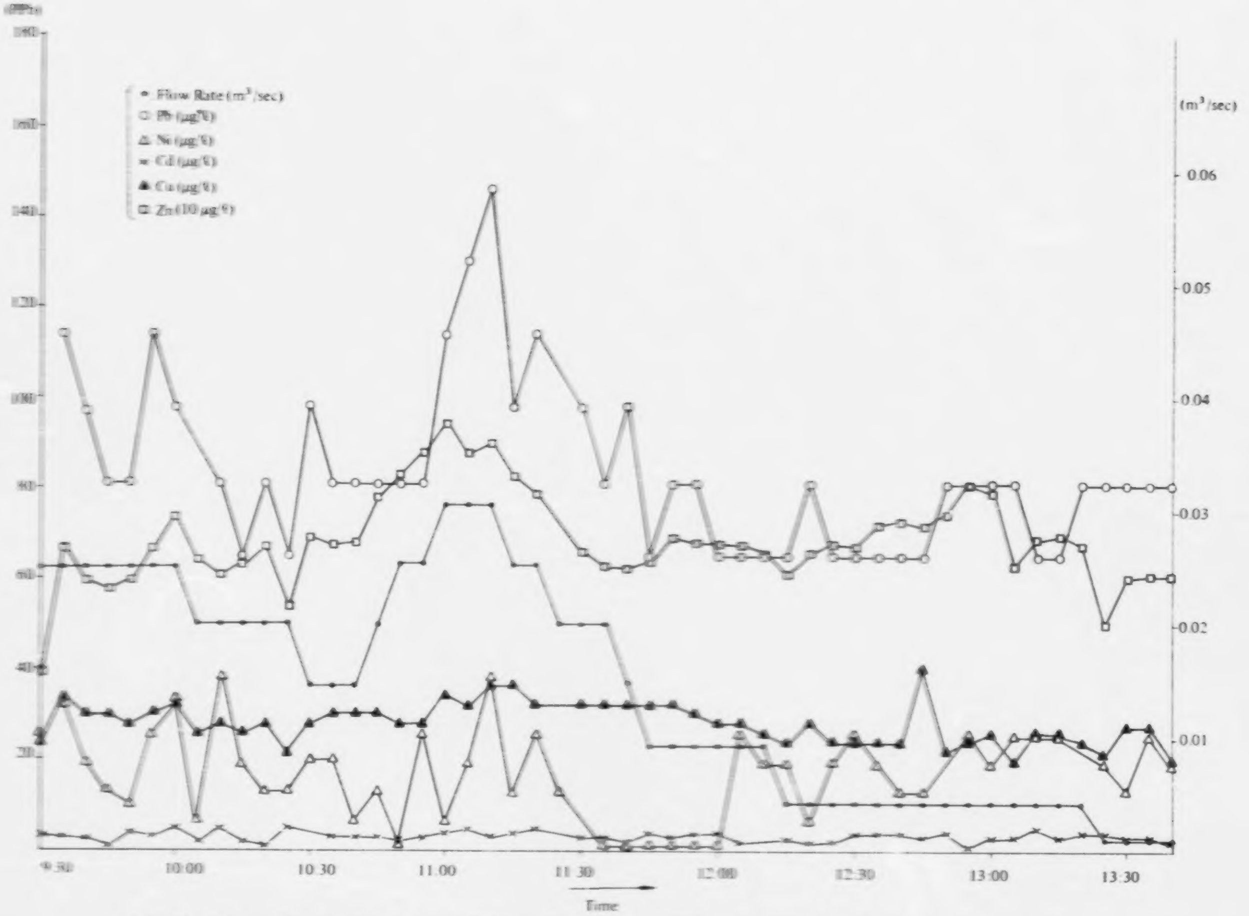


Fig. 5 Heavy Metals Contained in Urban Stormwater (22, Nov. 1974, Midorichyo, Yamagata)

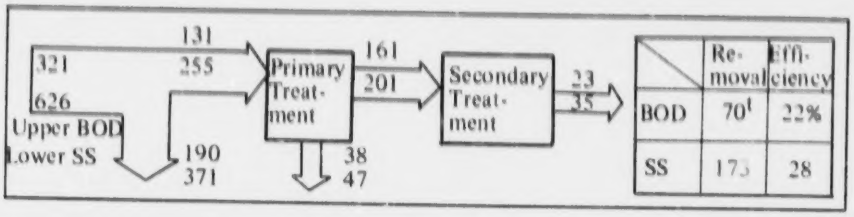

321
626
Upper BOD
Lower SS
131
255
190
371
Primary Treat-ment
161
201
38
47
Secondary Treat-ment
23
35
Re-moval
Effi-ciency
BOD
70t
22%
SS
173
28

Fig. 6 Annual Load of Pollutants from Combined Sewer Overflows, Yabatagawa, Tokyo. (t/Y)

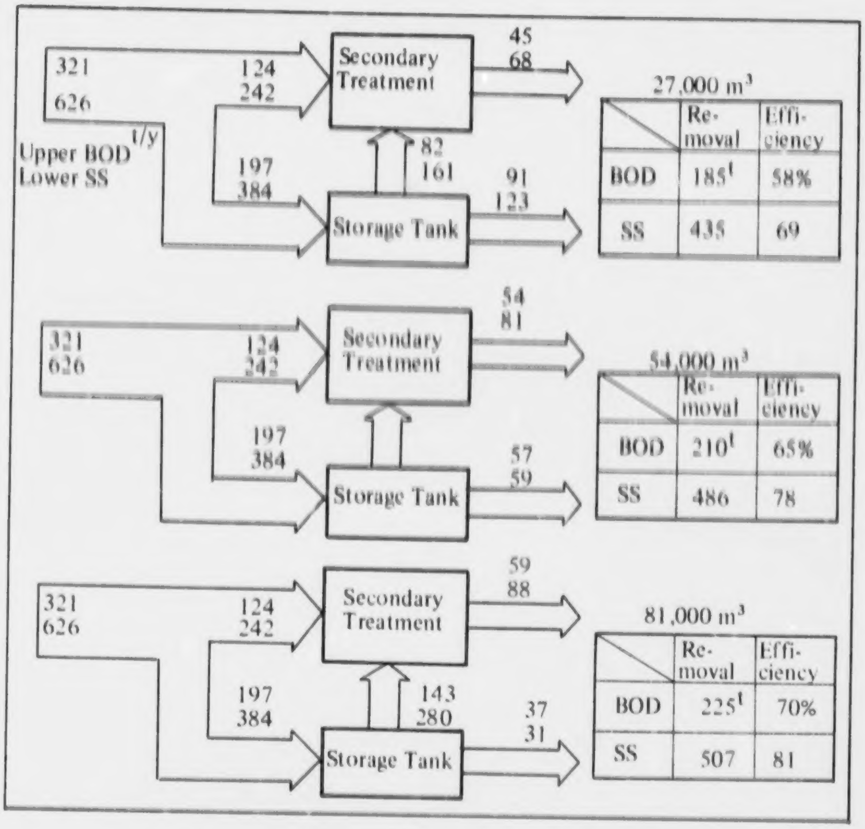

321
626
t/y
Upper BOD
Lower SS
124
242
Secondary Treatment
45
68
197
384
82
161
Storage Tank
91
123
27,000 m³
Re-moval
Effi-ciency
BOD
185t
58%
SS
435
69
321
626
124
242
Secondary Treatment
54
81
197
384
Storage Tank
57
59
54,000 m³
Re-moval
Effi-ciency
BOD
210t
65%
SS
486
78
321
626
124
242
Secondary Treatment
59
88
197
384
143
280
Storage Tank
37
31
81,000 m³
Re-moval
Effi-ciency
BOD
225t
70%
SS
507
81

Fig. 7 Estimated Effect of Storage Tank, Yabatagawa Tokyo

Fourth US/JAPAN Conference
on
Sewage Treatment Technology
Paper No. 8

CASE STUDIES OF INDUSTRIAL WASTEWATER TREATMENT

October 29, 1975
Washington, D. C.

Ministry of Construction
Japanese Government

CASE STUDIES OF INDUSTRIAL WASTEWATER TREATMENT

1. **Material Balance of Heavy Metals in Sewage Treatment Process** 420
   Mr. Ken Murakami, Chief, Water Quality Section, Water Quality Control Division, Public Works Research Institute, Ministry of Construction
2. **Torihama Industrial Wastewater Pretreatment Plant in Yokohama – Operation and Maintenance –** 451
   Mr. Masayuki Sato, Director, Sewage Works Bureau, City of Yokohama
3. **Fukashiba Industrial Wastewater Treatment Plant in Ibaragi Prefecture** 466
   Mr. Satoru Tohyama, Head, Sewage Works Division, Department of Sewarage & Sewage Purification, Ministry of Construction
4. **Design and Operation of Dying Waste Treatment Facility in Sabae City, Fukui Prefecture** 477
   Dr. Mamoru Kashiwaya, Head, Water Quality Control Division, Public Works Research Institute, Ministry of Construction
5. **Improvement of Effluent Quality of Bisai District Sewage Treatment Plant in Aichi Prefecture** 489
   Dr. Mamoru Kashiwaya, Head, Water Quality Control Division, Public Works Research Institute, Ministry of Construction

CHAPTER 1. MATERIAL BALANCES OF HEAVY METALS IN SEWAGE TREATMENT PLANTS

1.1 Effects of Cadmium, Lead and Mercury on the Biological Treatment Process 421
  1.1.1 Effects of Cadmium on the Biological Treatment 421
  1.1.2 Effects of Lead on the Biological Treatment 422
  1.1.3 Effects of Mercury on the Biological Treatment 423
  1.1.4 Summary 424
1.2 Influent Heavy Metal Concentration and Its Loading Balance in the Sewage Treatment Plants 425
  1.2.1 Sewage Treatment Plants Surveyed and Outline of Survey 425
  1.2.2 Results and Discussion 426
  1.2.3 Summary 427

1. MATERIAL BALANCES OF HEAVY METALS IN SEWAGE TREATMENT PLANTS

1.1 EFFECTS OF CADMIUM, LEAD, AND MERCURY ON THE BIOLOGICAL TREATMENT PROCESS

The Public Works Research Institute, Ministry of Construction has, during the past three years or so, conducted experiments using bench scale activated sludge apparatus to clasify effects of cadmium, lead, and mercury, on biological treatment process and material balance of heavy metals in the process.

The laboratory apparatus used is shown in Fig. 1.1. The apparatus was modified so that it can collect vaporized mercury when experimenting on mercury.

Table 1.1 shows the design data and loading factors of the experiments. Sewage used in the experiments is synthetic sewage and ingredients are shown in Table 1.2. Cadmium, lead, and mercury were added to sewage in the forms of cadmium sulfate, lead nitrate, and mercuric chloride, respectively.

The acclimatization period was taken about two weeks from the day the heavy metal addition was started.

1.1.1 EFFECTS OF CADMIUM ON THE BIOLOGICAL TREATMENT

i) Experiments was made at each of three levels of cadmium in the influent feed, 0.1, 1.0, and 10 mg/l.

ii) Average effluent quality obtained are shown in Table 1.3. In the case of influent with the cadmium concentration of 1 mg/l, no influence appeared on the effluent. But one containing 10 mg/l of cadmium experienced lowering of the transparency of the effluent and increase in SS and BOD. As for soluble BOD, no significant differences have been seen during all the experiments, but insoluble BOD in the effluent increased as cadmium concentration of sewage become higher.

From the relations between heavy metal content in activated sludge and those in SS of the effluent, which are shown in Table 1.4, it was conjectured that SS of the effluent is composed of organisms lighter than activated sludge, or their debris.

In the case of influent containing 1 mg/l of cadmium, cadmium concentration of the effluent was approximately 0.3 mg/l. Cadmium in the effluent was mostly a insoluble.

iii) Characteristics of activated sludge are shown in Table 1.5.

Relations between cadmium concentration in sewage sludge production and oxydation rate of sludge are shown in Fig. 1.2.

Transfer rate of removed BOD into sludge was about 0.5 when cadmium concentration was 1 mg/l, and about 0.7 when the concentration was 10 mg/l. These results may indicate that when heavy metals exist in sewage, excess activated sludge will increase.

Existence of cadmium in sewage has changed the fauna in activated sludge: protozoa died out and it became composed mostly of bacteria.

When cadmium concentration in the culture medium by batch culture is less than 10 mg/l, the growth rate of bacteria separated from activated sludge showed no significant difference against the control.

iv) Fig. 1.3 shows relations between cadmium content in activated sludge and cadmium concentration in the influent. This Figure clearly shows that cadmium content in activated sludge varies depending on cadmium concentration in the influent.

In 10 or so days after cadmium addition was started cadmium concentration in sludge became almost constant.

"Concentration factors" were approximately 28 x 10^3 when influent cadmium concentration was 0.1 mg/l and 1.0 mg/l, and 9 x 10^3 when it was 10 mg/l. These results were similar to those obtained in surveys at sewage treatment plants.

v) Table 1.6 shows the material loading balance of cadmium in the process. "Unaccounted for" in this Table is a value subtracted cadmium in effluent and inexcess activated sludge from that in influent. It is considered that this "unaccounted for" includes errors due to sampling of MLSS for analysis and those in measuring growth of activated sludge. In experiments on sewage containing 10 mg/l cadmium, pink-yellowish substance like cadmium sulfide was perceived on the wall of the aeration tank. This substance is also considered to be included in "unaccounted for."

Assuming that the "unaccounted for" flowed out as excess sludge, over 80% of it is considered to transfer into the sludge treatment process, when cadmium concentration is less than 1 mg/l.

As influent cadmium concentration increases, percentage of cadmium existing in effluent as insoluble form becomes higher. In the case of 1 mg/l of cadmium, about 80% of cadmium flowing out together with effluent was insoluble.

1.1.2 EFFECTS OF LEAD ON THE BIOLOGICAL TREATMENT

i) Experiments was made at each of five levels of lead in the influent feed, 0.1, 0.25, 1.2, 7.9, and 95.5 mg/l.

ii) Average effluent quality obtained are shown in Table 1.7. When lead concentration in influent was 1 mg/l, no influence appeared on the effluent. When the concentration was 8 mg/l, transparency of effluent became lower and the quantity of SS and insoluble BOD increased. Significant differences in the soluble organic constituents in effluent were hardly perceived in each run.

From Table 1.4, it has been found that SS in effluent was due to the washout of activated sludge.

Effects of lead on the biological treatment appeared when its concentration in influent exceeds 8 mg/l. Symptom is a washout of activated sludge.

iii) Characteristics of activated sludge are shown in Table 1.8.

As in the case of cadmium, sludge production tended to increase by the addition of lead.

In the case of transfer rate of removed BOD into sludge, the results were similar to those in experiments with cadmium. When lead concentration in influent was less than 8 mg/l, transfer rate of removed BOD into sludge was 0.3 to 0.4. As for fauna in activated sludge, when influent contained 8 mg/l of lead, *Volticella, Epistylis* and *Charchesium* were perceived in a small quantity.

Toxicity of lead was examined by shaking culture using activated sludge in which protozoa does not exist. As a result, when the concentration of lead was less than 100 mg/l, the activity of activated sludge deteriorated temporarily. But the recovery rate of activity of activated sludge was at the same level of that of control.

iv) Fig. 1.3 shows relations between content of lead in activated sludge and lead concentration in the influent.

In about a week after lead was added to the influent, the lead concentration in activated sludge became a constant thus reaching a steady state.

The concentration factor was 8.8 to 11.1 x 10^3 when lead in influent was less than 8 mg/l.

Because lead tends to produce insoluble compounds when mixed with influent, it is considered that the state of lead in sludge is different from that of other elements.

v) Table 1.6 shows the material loading balance of lead in the process. "Unaccounted for" was similarly treated as with the case of cadmium. The quantity of lead transferring into the sludge treatment process tended to increase as the lead concentration in influent increased.

When the lead concentration in influent was less than 1 mg/l, 30 to 50% of lead in effluent was discharged as insoluble form. And when the concentration was 8 mg/l, over 90% of lead flowed out was insoluble.

1.1.3 EFFECTS OF MERCURY ON THE BIOLOGICAL TREATMENT

i) Experiments was made at each of three levels of mercury in the influent feed, 4, 100 and 1,000 mg/l.

ii) Average effluent quality obtained are shown in Table 1.9. When mercury in effluent was 4 μg/l, SS, COD, etc. of effluent increased and transparency decreased.

As in the case of cadmium, SS in effluent is considered as organisms lighter than activated sludge or their debris (Table 1.4).

It seems that effects of mercury on the biological treatment are observed when the mercury in influent exceeds 4 μg/l.

iii) Table 1.10 shows characteristics of activated sludge. Sludge production showed the same tendencies as with cadmium and lead.

Transfer rate of removed BOD into sludge was 0.4 when mercury in influent was 4 μg/l and about 0.5 when it was 1 mg/l. The same rate of control was about 0.3.

When mercury in influent was less than 4 μg/l, protozoa was perceived in activated sludge. But when the mercury concentration was 100 μg/l, bacteria similar to zoogloea were dominant.

iv) In the biological treatment, when mercury in influent was about 1 mg/l, the

amount of vaporized mercury was aproximately 17 μg/day/g.SS. In other runs with mercury, quantitative analysis of vaporized mercury was impossible.
State of vaporization of mercury in aeration tanks is shown Fig. 1.4. As clearly shown in the Figure, it has been found that most of mercury to be vaporized in aeration tanks is vaporized at the first quarter of the path of the tanks.

v) Relations between mercury content in activated sludge and mercury concentration in influent are shown in Fig. 1.3.
As a result of measurement by a gaschromatograph whose sensitivity is 0.01 mg alkyl-mercury/kg.SS alkyl-mercury was not detected in activated sludge.
Concentration factor was 42×10^3 when mercury in influent was about 4 μg/l.

vi) Table 1.6 shows material loading balance of mercury in the process.
Unlike in the cases of cadmium and lead, when mercury in influent was about 4 μg/l, the total of mercury discharged as effluent and vaporized mercury was some 10%. Therefore, mercury is easily accumulated in sludge.
Around 50% of effluent mercury was insoluble.

1.1.4 SUMMARY

1) When cadmium, lead, or mercury is contained in sewage, the minimum concentration of them which gives effects on the biological treatment is as follows:

Cadmium		1 to 10 mg/l
Lead	approx.	10 mg/l
Mercury	approx.	5 μg/l

Actual effects of these heavy metals on the biological treatment appeared in the form of decrease in transparency of effluent and increase in SS.
Within the above-mentioned level of heavy metal concentration, there were virtually no effect of them on removal of organic matters.

2) In cases of cadmium and mercury, SS in effluent showed those characters which no protozoa existed in activated sludge. In the case of lead, SS in effluent was due to the washout of activated sludge.

3) By the addition of heavy metals, the fauna of activated sludge was changed: while protozoa decreased in number, bacteria became dominant species.
Within ranges of concentration of heavy metals in the present experiments, growth rate of bacteria separated from activated sludge was hardly affected.

4) Heavy metals in influent tended to accelerate sludge production. This is considered, from 3) above, to be caused by the difference in food chain of organism in activated sludge.

5) Accumulation of heavy metals on activated sludge depends upon their concentration in influent.

6) Concentration factor of the heavy metals was around 10^4. Most of heavy metals contained in influent is transferred to the sludge treatment process.

7) Outline of effects of each element on the biological treatment process is shown in Table 1.11.

1.2 INFLUENT HEAVY METAL CONCENTRATION AND ITS LOADING BALANCE IN THE SEWAGE TREATMENT PLANTS

In Japan, the concern in the effects of heavy metals in the sewage upon the activated sludge process is veering round toward the extent to which the heavy metals, though little in quantity, in the sewage are entrained into the sludge through the biological treatment process, rather than toward the interference with the biological treatment itself.

This is because as the environmental standards and pretreatment facilities effluent standards hav een established it is almost unlikely so far as municipal sewage treatment plants are concerned that heavy metals will run into them at as high concentrations as to interfere biological treatment.

On the other hand, it is noticed that even if heavy metals in the influent is a trace level, they can accumulate to amass their concentrations in the sludge. This fact urges the collection of information about the relationship between the concentrations of heavy metals in the influent sewage and those in the sludge.

Dealt with here are the concentrations of heavy metals contained in domestic sewage, concentrations of heavy metals accumulated in sludge and heavy metal balance in the sewage treatment plant which are discussed based on the results of survey on the sewage treatment plants treating domestic sewage alone and those taking in industrial effluents combined with domestic sewage.

1.2.1 SEWAGE TREATMENT PLANTS SURVEYED AND OUTLINE OF SURVEY

A field survey was conducted from July 1973 to September 1974. The sewage treatment plants surveyed included two domestic sewage treatment plants located near residential areas and two comparatively large municipal waste treatment plants.

The latter two were picked up as typical of those treating municipal sewage and not as special problem solvers of industrial wast water.

These four sewage treatment plants are outlined in Table 2.1.

Plant "A" is a medium-sized one which is estimated to have been receiving some 8,000 m^3/d from industries suspected to discharge heavy metals out of its total daily processing rate of 50,000 m^3.

Plant "B" comes under the catagory of large-scale plants. It has been treating 400,000 m^3/d of which some 200,000 m^3/d are accounted for by industrial wastewater, and is estimated to have been receiving at least about 2,000 m^3/d from industries likely to discharge heavy metals.

Plants "C" and "D" are small-scale ones in the residential areas which are not receiving industrial wastes at all.

All these plants are undertaking the conventional activated sludge process after primary treatment. The flow diagram is all the same.

Plants "A" and "D" have their sludge treated at nearby sludge treatment plants. Plants "B" and "C" are practising anaerobic digestion of sludge, and the supernatant to be developed in the sludge treatment process is returned to the head of the primary settler.

The survey was conducted with one day as a unit. The sampling was conducted for 24 consecutive hours; some samples were gathered into a composite one, and some others were handled as grab samples.

1.2.2 RESULTS AND DISCUSSION

Concentrations of heavy metals in domestic sewage

Table 2.2 shows the mean values and range of concentrations of heavy metals in the domestic sewage obtained at plants "C" and "D".

Copper, chromium, cadmium, lead, nickel, zinc and mercury were measured. As shown in Table 2.2, all these seven heavy metals were detected.

The concentration of zinc was the highest with the order of 10^{-2} to 10^{-1}, followed by copper with 10^{-2}, chromium, nickel and lead with 10^{-3} to 10^{-2}, cadmium with 10^{-3} and mercury with 10^{-4}, all in terms of mg/lit.

From what these heavy metals come is still unknown, but foods, plumbings, kitchen utensils and the like are suspected. It is also reported that cadmium and mercury are deeply concerned with foods.

Accumulation of heavy metals in sludge

Primary sludge and waste activated sludge sampled from each plant were measured for heavy metals.

Figs. 2.1 and 2.2 show the relationships between the concentrations of heavy metals in the influent sewage and those in the primary sludge and waste activated sludge.

For both, daily average values are taken. It is evident from the figures that the concentrations in the influent have a great bearing on the concentrations in the primary sludge and waste activated sludge. The straight lines appearing in the figures show the multiples (accumulation coefficients) of the concentrations in influent and sludge.

The concentrations of heavy metals in the influent sewage were as small as less than 1 mg/lit., and the laboratory test results explained in Chap. 1, and the results of field survey and laboratory tests conducted by Taft Center, U.S.A. are annexed in order to corroborate the above findlings. (See Figs. 2.3 and 2.4.)

As a result, there are found the interesting facts as follows.

1) In the case of primary sludge, when the concentrations of heavy metals in the influent sewage exceed 10 mg/lit., the relationship between the concentrations in the influent and those in the sludge loses linearity, with the result that the concentrations in the sludge become in a specified range (1 to 10 mg/SSg)

2) In the case of waste activated sludge, even when the concentrations in the influent sewage go up to 100 mg/lit., it does not matter to the linearity; namely, the concentrations of heavy metals in the waste activated sludge increase on and on.

Considering these and other various factors, the following may be said about the accumulation of heavy metals in the sludge in the sewage treatment plant.

1) At sewage treatment plants operated under ordinary conditions, the concentra-

tions of heavy metals in the sludge are closely related with those in the influent sewage, though dependent on the kinds of heavy metals.

2) The multiples (accumulation coefficients) lie in the range of 10^3 to 10^4, and 10^4 should be taken up if safety factor is considered.

3) In the case of primary sludge, the linear relationship between the concentrations of heavy metals in the influent sewage and those in the sludge is lost when the former exceeds 10 mg/lit., and thus the concentrations of heavy metals in the sludge remain within the range of 1 to 10 mg/ssg.

4) The field surveys at sewage treatment plants could not clarify the difference in accumulation rate between metals.

5) The sludge delivered from the sewage treatment plants processing domestic sewage alone also concentrate heavy metals. According to the measurements at plants "C" and "D", the heavy metals are estimated to have been concentrated to the levels given below.

   Copper: 100 ~ 1,000 mg/kg; zinc: 100 ~ 10,000 mg/kg; chronium, nickel & lead: 10 ~ 1,000 mg/kg; cadmium: 10 ~ 100 mg/kg; mercury: 1 ~ 10 mg/kg.

Heavy metals balance in the sewage treatment plant

As regards plant "B" where supernatant is sent back to the head of the primary settler from the sludge treatment system, the results of heavy metals balance between facilities are shown in Table 2.3.

The figures are all based on actual observation. The run-in and run-off values of each facility are given with the total amount of heavy metals in the influent sewage taken as 100.

Namely, run-in and run-off amounts of each facility are divided by the amount in the influent sewage. The following are found by twice surveys.

1) Nickel and cadmium are liable to remain in the effluent. As regards cadmium, this tendency is considered attributable to its relatively low concentration in the influent sewage.

2) The amount of heavy metals conveyed by recycled waste such as supernatant of anaerobic digester is by far the more larger than that in the influent sewage; in some cases, it becomes 4 times as large.

3) Almost all of heavy metals contained in the recycled waste are insoluble.

4) Accordingly, they are settled easily in the primary settler, and are recycled between the sludge treatment system and sewage treatment system. The concentrations of heavy metals in the influent sewage become almost equal to those in the primary effluent.

1.2.3 SUMMARY

1) Domestic sewage contains heavy metals, whose concentrations are found to be in certain ranges.

2) It is disclosed that the accumulation of heavy metals in the sludge is closely related to the concentrations of heavy metals in the influent sewage.

3) The accumulation coefficient is in the order of some 10^4 irrespective of the kinds of heavy metals, and when the concentrations in the influent sewage are once clarified, the concentrations in the sludge can therefore be estimated.

4) An actual example of material balance concerning heavy metals in the sewage treatment plant is given.

Table 1.1 Design Data and Loading Factor (Bench scale activated sludge apparatus)

Apparatus design data and loading factors		
Aeration tank		
Capacity	(*l*)	60
Aeration period	(h)	4
Return sludge rate	(%)	25
BOD loading	(kg/day/kg. ss)	0.3
Final settler		
Capacity	(*l*)	24
Detention time	(h)	1.6
Surface overflow rate	(*l*/day/cm^2)	0.87

Table 1.2 Composition of Synthetic Sewage (mg/*l*)

Substrate	Concentration	Substrate	Concentration
Glucose	17.6	Urea	37.0
Dextrin	17.6	KC*l*	8.1
Meat extract	80.0	NaC*l*	8.1
Peptone	80.0	$MgSO_4$	5.4
Yeast extract	90.0	$KH_2 PH_4$	24.5

BOD ≒ 180 mg/*l*

Table 1.3 Average Characteristics of Effluents for Control and Cadmium-fed Apparatus

S: Soluble matter

No. (Cd in influent)	Trans.	PH	SS (mg/ℓ)	BOD			Cd		
				(mg/ℓ)	S(mg/ℓ)	Percent removal	(mg/ℓ)	S(mg/ℓ)	Percent removal
A (0 mg/ℓ)	30 <	5.8 ~ 6.9	36.3 16 ~ 104	8.68	0.96	94.2	–	–	–
B (0.1)	13 ~ 30 <	5.6 ~ 6.6	50.7 2 ~ 140	6.96	1.09	95.2	0.022	0.013	80.0
C (1.0)	11 ~ 30 <	5.6 ~ 7.0	51.0 10 ~ 108	6.94	0.57	95.2	0.30	0.049	71.1
D (12.4)	21 15 ~ 25	6.8 ~ 7.3	82.4 34 ~ 144	12.4	0.99	91.4	6.91	1.04	44.2

Table 1.4 **Metal Content in Activated Sludge and SS of Effluent**

Metal	No. (Metal in influent)		Content of metal in sludge (mg/g. ss)		
			Aa	Ab	Aa/Ab
Cd	A	(0 mg/ℓ)	–	–	–%
	B	(0.11)	3.08	0.2	6.5
	C	(1.04)	29.6	4.9	16.6
	D	(12.4)	113	71.2	62.8
Pb	A	(0)	–	–	–
	B	(0.1)	0.99	0.85	85.9
	C	(0.25)	2.23	1.95	87.4
	D	(1.2)	13.3	12.7	95.6
	E	(7.9)	61.6	256	–
	F	(95.5)	534	378	70.8
Hg	A	(0)	–	–	–
	B	(0.004)	0.187	0.022	11.8
	C	(0.974)	13.2	0.082	0.6

Aa: Metal content in activated sludge, Ab: Metal content in SS of effluent

Table 1.5 Effect of Cadmium on Activated Sludge

No. (Cd in influent, mg/ℓ)	MLSS (mg/ℓ)	MLVSS (mg/ℓ)	MLVSS /MLSS	SVI	BOD loading (kg/day/kg. ss)	Cd loading (mg/day/g. ss)	Sludge production rate (g/day)		Content of Cd in sludge (mg/g. ss)	Concentration factor
							as SS	as VS		
A (0)	3,814	3,443	0.90	214	0.23	–	19.4	17.5	–	–
B (0.1)	2,818	2,519	0.89	289	0.31	0.17	24.6	21.9	3.08	28×10^3
C (1.0)	2,957	2,567	0.87	239	0.30	2.1	27.9	24.3	29.6	28×10^3
D (12.4)	2,702	2,294	0.85	283	0.32	27.5	38.7	32.9	113	9×10^3

Table 1.6 Balance of Metals in Activated Sludge Process

Metal	No. (Concentration in influent)	Material balance (%)			
		Effluent	Vapor	Excess activated sludge	Unaccounted for
Cd	B (0.1 mg/l)	20.1	–	51.3	28.6
	C (1)	28.8	–	78.3	–
	D (10)	55.8	–	23.1	21.1
Pb	B (0.1)	55.6	–	22.2	22.2
	C (0.25)	27.3	–	41.5	31.2
	D (1.2)	32.6	–	29.1	38.3
	E (7.9)	81.7	–	19.8	–
	F (95.5)	10.6	–	44.7	44.7
Hg	B (4.4 Pμg/ℓ)	5.27	4.11	62.3	28.3
	C (119)	1.57	1.27	83.6	13.6
	D (974)	0.34	0.31	89.0	10.4

Table 1.7 Average Characteristics of Effeuents for Control and Lead-fed Apparatus

No.	Trans.	PH	SS	BOD			COD			Pb		
(Pb in influent)			(mg/ℓ)	(mg/ℓ)	S (mg/ℓ)	Percent removal	(mg/ℓ)	S (mg/ℓ)	Percent removal	(mg/ℓ)	S (mg/ℓ)	Percent removal
A (0 mg/ℓ)	29.6	6.9	7.60	7.05	2.21	95.3	9.22	6.17	86.2	–	–	–
B (0.1)	30 <	7.0	21.2	12.26	1.73	91.5	11.83	7.12	73.9	0.056	0.038	43.6
C (0.25)	30 <	7.3	12.8	8.04	1.49	94.6	7.27	5.62	84.6	0.070	0.045	29.7
D (1.2)	30 <	7.0	14.2	8.10	1.68	94.5	7.31	5.71	88.2	0.39	0.21	67.5
E (7.9)	26.1	6.4	23.2	15.25	1.50	88.7	9.92	5.55	84.6	6.47	0.54	18.3
F (95.5)	20.0	6.7	25.7	(5.16)	(1.32)	(96.4)	7.60	4.82	87.2	10.08	0.36	89.4

Table 1.8 Effect of Lead on Activated Sludge

No. (Pb in influent, mg/ℓ)	MLSS (mg/ℓ)	MLVSS (mg/ℓ)	MLVSS /MLSS	SVI	BOD loading (kg/day/kg. ss)	Pb loading (mg/day/g. ss)	Sludge production rate (g/day)		Content of Pb in sludge (mg/g. ss)	Concentration factor
							as SS	as VS		
A (0)	2,419	2,201	0.91	318	0.64	–	9.4	8.5	–	–
B (0.1)	2,263	1,969	0.87	90	0.40	0.273	14.9	13.0	0.99	9.8×10^3
C (0.25)	2,280	1,987	0.87	95	0.40	0.700	21.2	18.5	2.23	8.8×10^3
D (1.2)	1,474	1,340	0.91	480	0.59	5.90	14.4	13.1	13.28	11.1×10^3
E (7.9)	1,751	1,384	0.80	123	0.56	308	17.3	13.9	61.6	9.8×10^3
F (95.5)	12,860	3,342	0.27	30	0.08	476	22.0	6.0	534	5.6×10^3

Table 1.9 Average Characteristics of Effluents for Control and Mercury-fed Apparatus

No. (Hg in influent)	Trans.	PH	SS	BOD			COD			Hg		
			(mg/ℓ)	(mg/ℓ)	S(mg/ℓ)	Percent removal	(mg/ℓ)	S(mg/ℓ)	Percent removal	(μg/ℓ)	S(μg/ℓ)	Percent removal
A (0 μg/ℓ)	30	7.0	3.5	4.42	0.98	96.8	6.43	4.97	88.9	–	–	–
B (4.4)	21.3	7.3	25.5	8.13	2.05	93.9	12.30	5.71	79.2	0.97	0.41	78.3
C (974)	20.3	7.3	20.0	18.40	3.66	85.6	18.50	7.85	67.7	3.35	1.72	99.7
A' (0)	30	7.0	–	4.55	0.79	95.3	8.68	7.81	87.0	–	–	–
D (119)	30	7.0	–	4.57	0.70	95.3	18.33	9.11	63.4	1.87	1.34	98.4

Table 1.10 Effect of Mercury on Activated Sludge

No. (Hg in influent, μg/ℓ)	MLSS (mg/ℓ)	MLVSS (mg/ℓ)	MLVSS /MLSS	SVI	BOD loading (kg/day/kg. ss)	Mercury loading (μg/day/g. ss)	Sludge production rate (g/day)		Content of Hg in sludge (μg/ss. g)	Concentration factor
							as SS	as VS		
A (0)	1,351	1,174	0.87	644	0.57	–	15.9	13.8	3.22	–
B (44)	1,676	1,473	0.88	482	0.47	15.98	31.4	27.6	187	42×10^3
C (974)	1,065	956	0.90	688	0.75	5,490	30.8	27.7	1.34×10^4	14×10^3
A' (0)	2,077	1,830	0.88	421	0.21	–	10.3	9.1	3.05	–
D (119)	1,437	1,236	0.86	324	0.24	410	12.5	10.8	2.87×10^3	24×10^3

Table 1.11 Summary of the Result

Metal	Expected maximum concentration in influent	Highest dose of metal for satisfactory biological treatment	Percent removal		Content of metal in sludge (mg/ss. g)	Growth of sludge μ (1/day)
			BOD	Metal		
Cd	0.1 mg/ℓ	1 ~ 10 mg/ℓ (5)	95	80	3 Concentration factor (28×10^3)	0.1455 Control (0.0848) 1.72*
Pb	1.0	10	95	70	13 (11×10^3)	0.1629 (0.0544) 2.53*
Hg	5 μg/ℓ	>5 μg/ℓ	94	80	0.2 (42×10^3)	0.3125 (0.1957) 1.60*

*: Ratio against control

Table 1.12 Outline of STP Surveyed

Item \ Name of STP	A	B	C	D
Area served (ha)	1,134	2,939	701	748
Population served	139,000	498,600	6,000	46,000
Daily average flow (m^3/d)	53,000	400,000	4,000	20,000
Collection system	Combined sewer	Combined sewer	Separate sewer	Separate sewer
Secondary treatment system	Conventional activated sludge	The same as left	The same as left	The same as left
Remarks			Domestic sewage	Domestic sewage

Table 1.13 Heavy Metal Concentration in Domestic Sewage

(mg/ℓ)

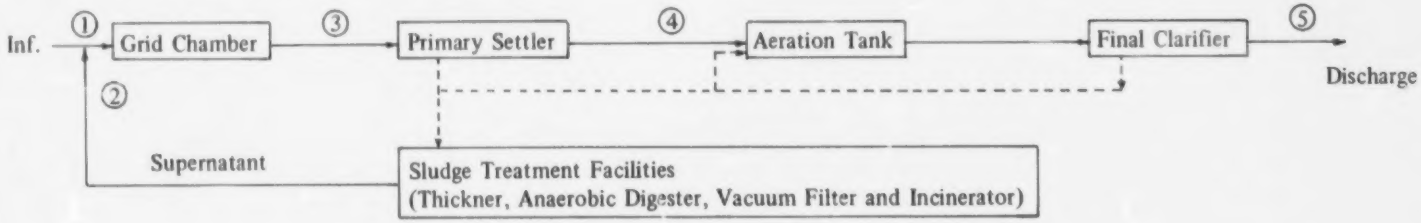
Heavy metal / STP	Cu	Cr	Cd	Pb	Ni	Zn	Hg	Remarks
D, Feb. '75	0.029	0.001	0.0012	0.0025	0.009	0.13	–	3 day average
D, Mar. '75	0.029	0.000	0.0029	0.0013	0.000	0.12	–	3 day average
D, Feb. '73	0.046	0.018	0.0022	0.020	0.055	0.295	0.0000	A day composit
D, Nov. '73	0.031	0.000	0.0016	0.016	0.001	0.156	0.0001	A day composit
C, Sept. '73	0.029	0.023	0.0026	0.038	0.019	0.055	0.0002	A day composit
C, Mar. '73	0.016	0.032	0.0031	0.028	0.015	0.054	0.0005	A day composit

Table 1.14 Heavy Metal Loading Balance in Sewage Treatment Plant (Observed at B STP)

Heavy metal / Location	Cu			Cr			Cd			Pb			Zn			Ni		
	Total	Sol.	Insol.	Total	Sol.	Insol.	Total	Sol.	Insol.	Total	Sol.	Insol.	Total	Sol.	Insol.	Total	Sol.	Insol.
1 Raw sewage	100	46	54	100	85	15	100	–	–	100	9	91	100	37	63	100	83	17
	100	55	45	100	48	52	100	82	18	100	27	73	100	16	84	100	45	55
2 Supermatant	478	1	477	301	1	300	423	28	390	310	7	303	442	2	441	222	13	209
	16	1	15	182	3	180	175	9	166	147	4	143	390	1	390	60	5	55
3 Primary Influent	519	. 11	508	347	41	306	253	231	22	420	9	411	483	3	449	325	94	232
	129			300	37	260	233	137	95	265	7	258	365	14	350	41	27	15
4 Primary Effluent	103	13	90	159	46	113	219	118	101	113	0	113	111	31	80	95	90	5
	90	16	74	124	32	92	144	79	65	70	32	38	78	16	61	56	45	11
5 Final Effluent	13	11	2	29	29	0	294	–	–	21	–	–	28	–	–	110	97	13
	17	<1	17	23	<1	23	67	<1	67	9	0	9	24	<1	24	45	1	44
Concentration in raw sewage (mg/ℓ)	0.137			0.113			0.0039			0.048			0.275			0.041		
	0.111			0.105			0.0027			0.035			0.211			0.040		

Notes: 1. Number indicates raw sewage values of 100 or proportional relationship based on it.

2. Above Aug. Survey, Beneath, Mar. Survey.


Inf.
①
Grid Chamber
②
③
Primary Settler
④
Aeration Tank
Final Clarifier
⑤
Discharge
Supernatant
Sludge Treatment Facilities
(Thickner, Anaerobic Digester, Vacuum Filter and Incinerator)

Schematic Flow Diagram of the B STP

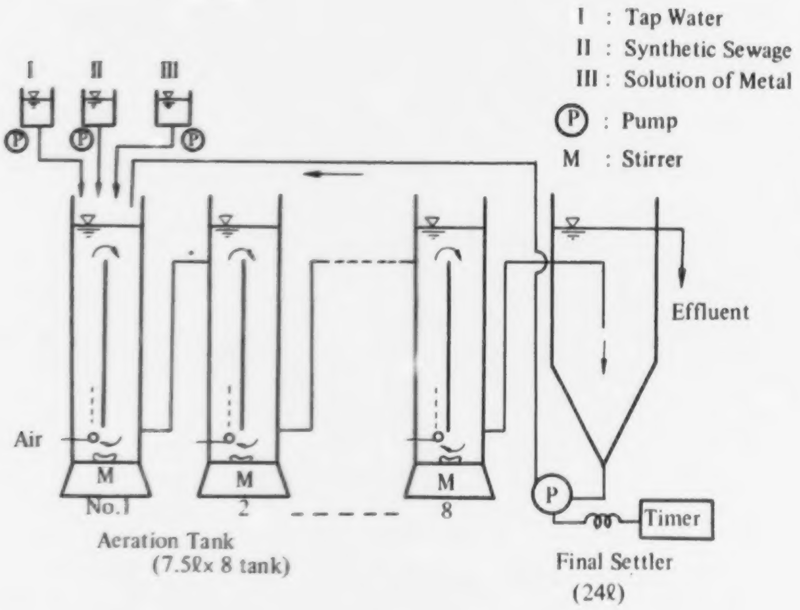

I : Tap Water
II : Synthetic Sewage
III : Solution of Metal
P : Pump
M : Stirrer
I
II
III
P
P
P
Air
M
M
M
No.1
2
8
Aeration Tank
(7.5ℓx 8 tank)
Effluent
P
Timer
Final Settler
(24ℓ)

Fig. 1.1 Bench Scale Activated Sludge Apparatus

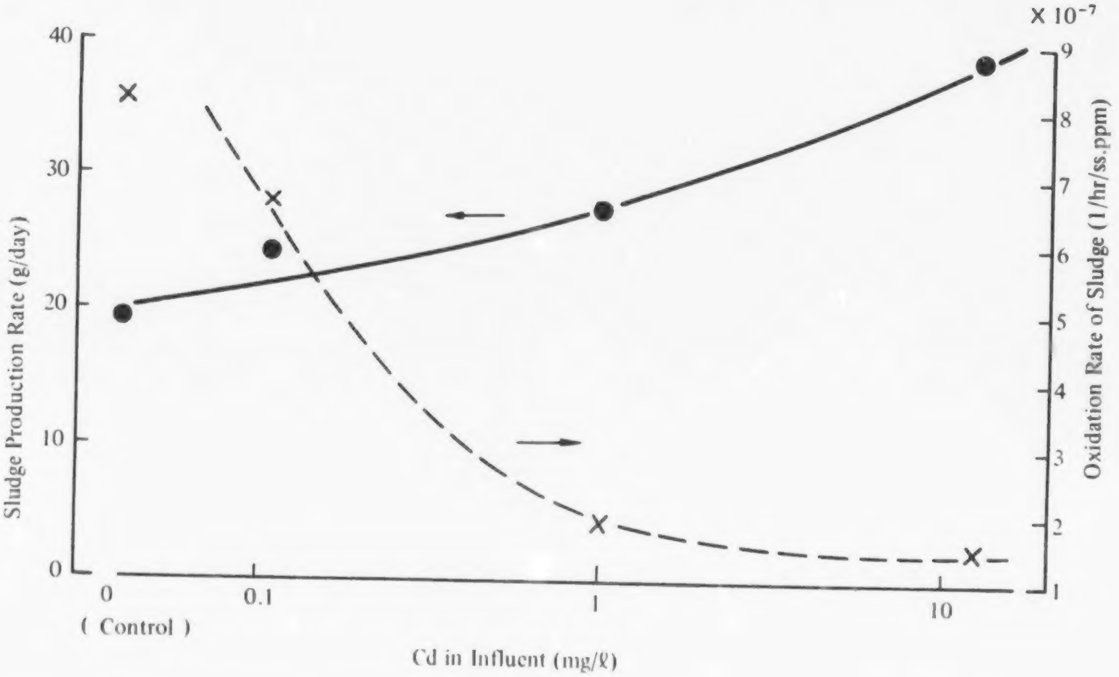


Fig. 1.2 Relationship Between Sludge Production and Oxidation Rate and Cd in Influent

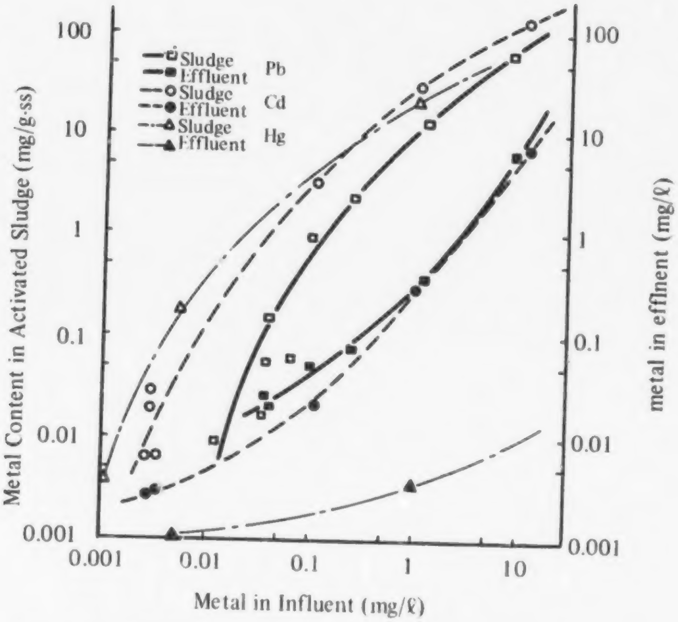


Fig. 1.3 **Rerationship between Influent and Effluent Heavy metal Concentration and Content in activated Sludge**

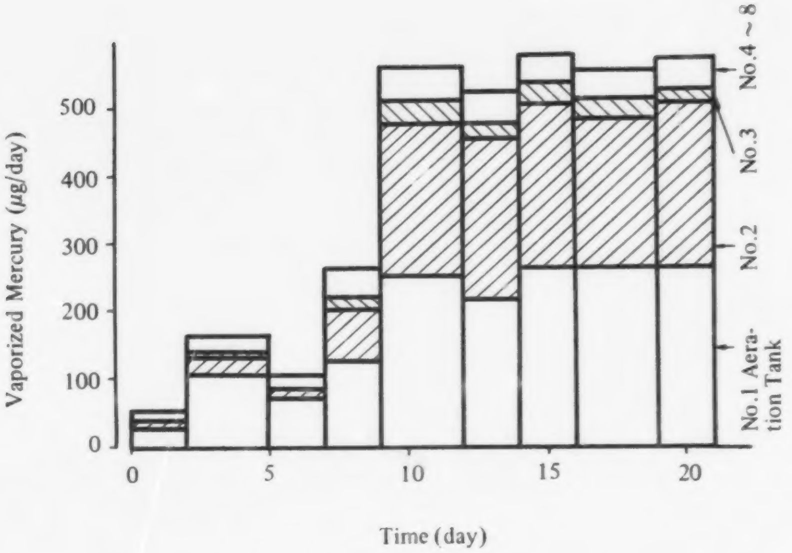


Fig. 1.4 Vaporized Mercury in Each Aeration Tank

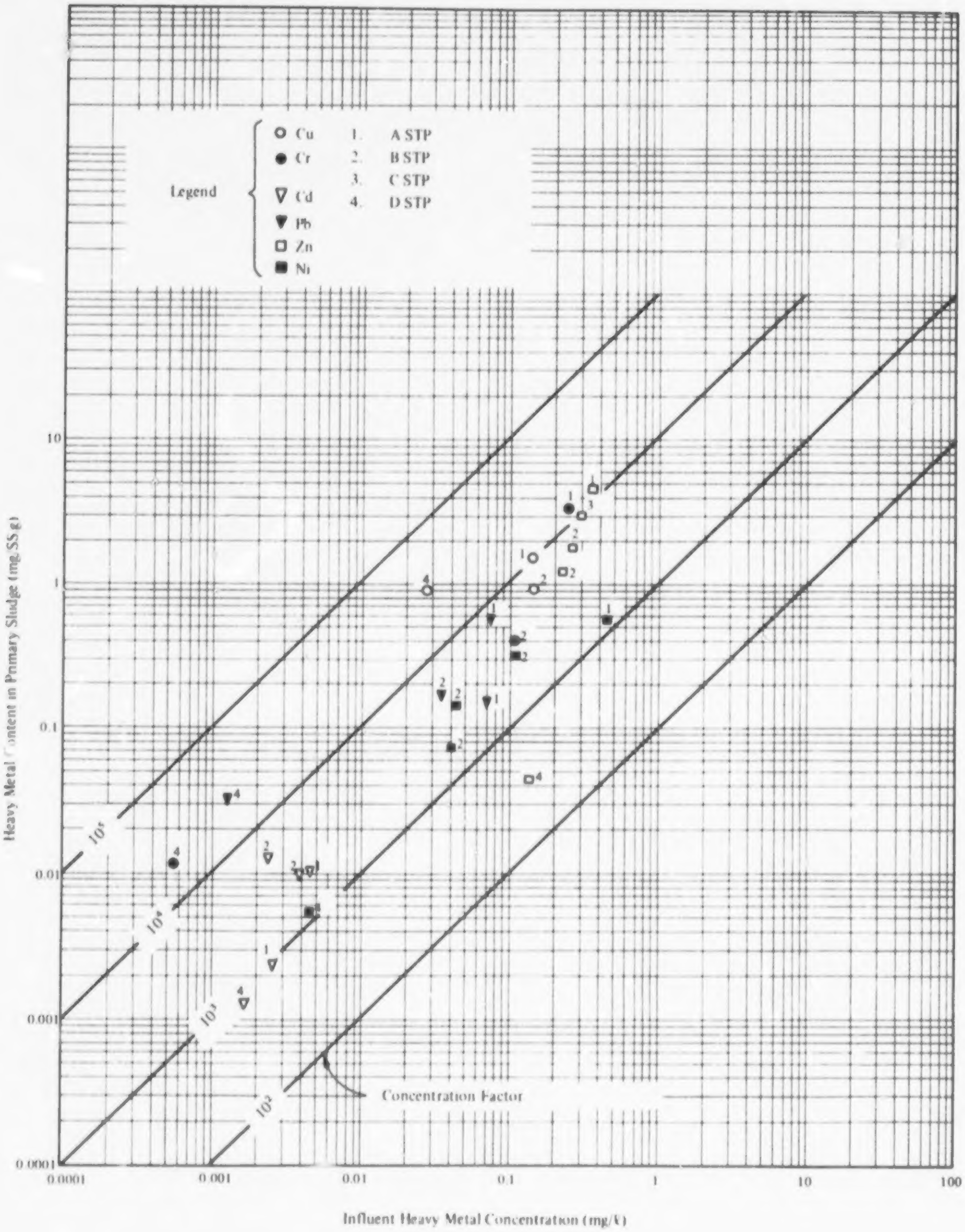


Fig. 1.5 Relationship between Influent Heavy Metal Concentration and Content in Primary Sludge

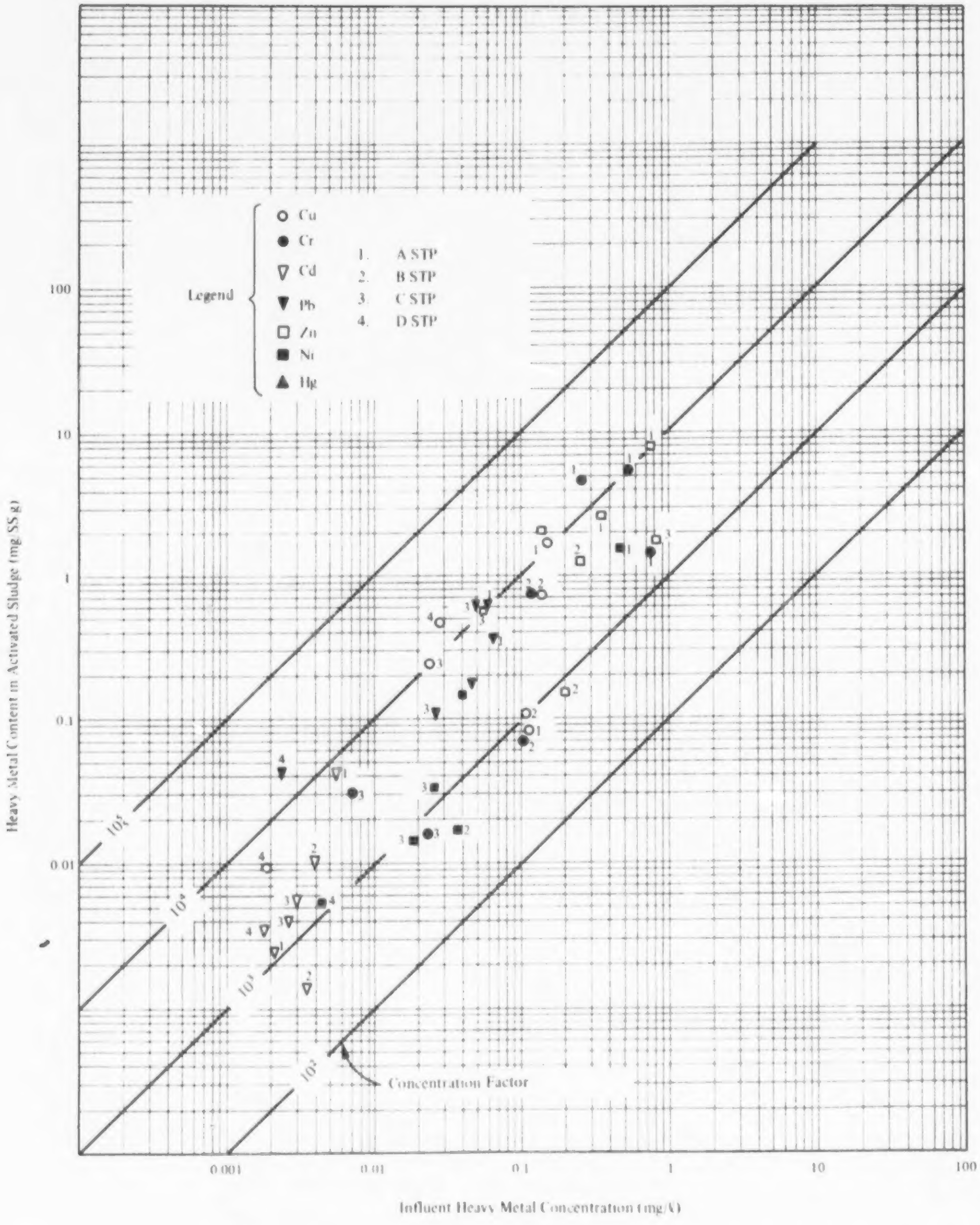
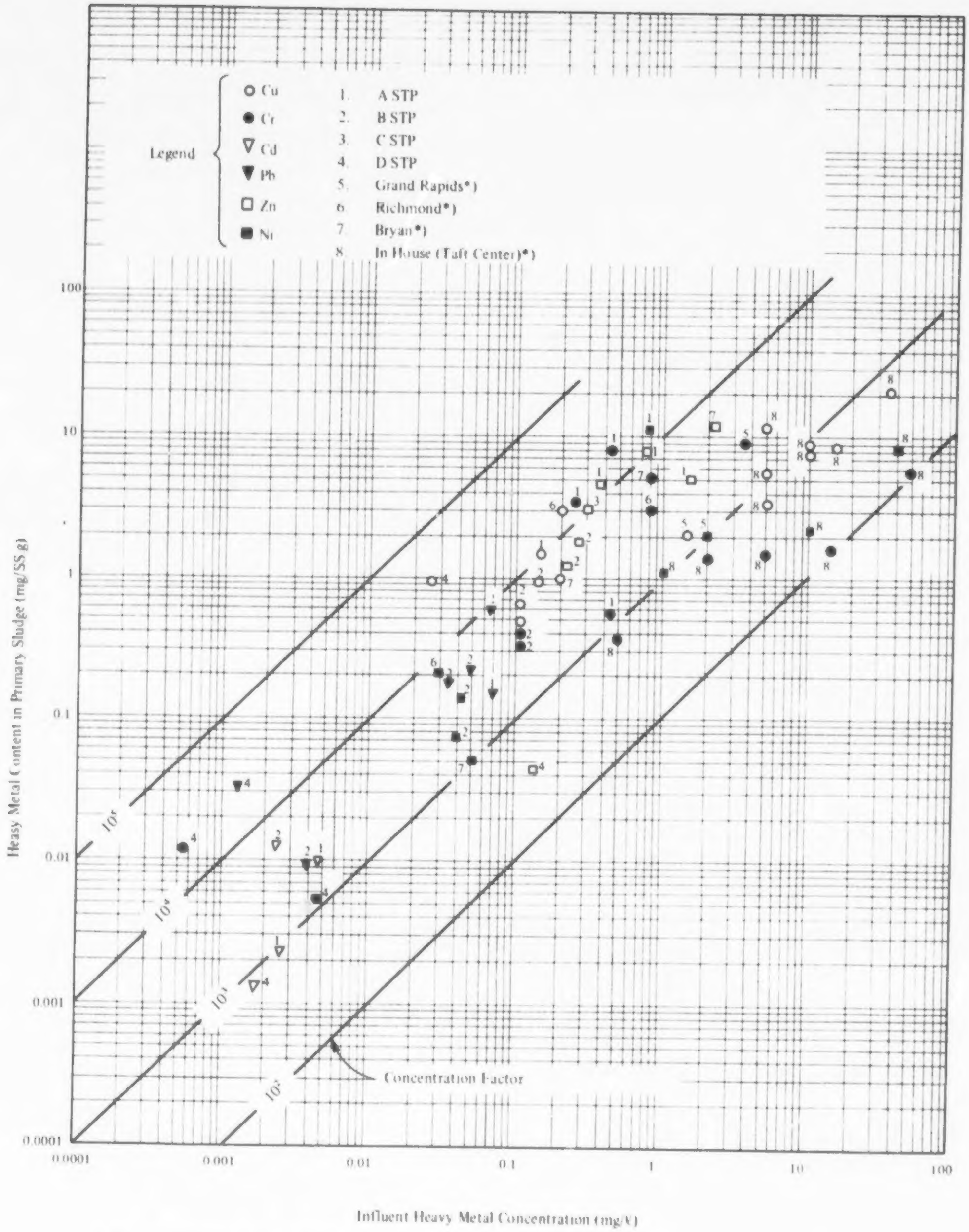
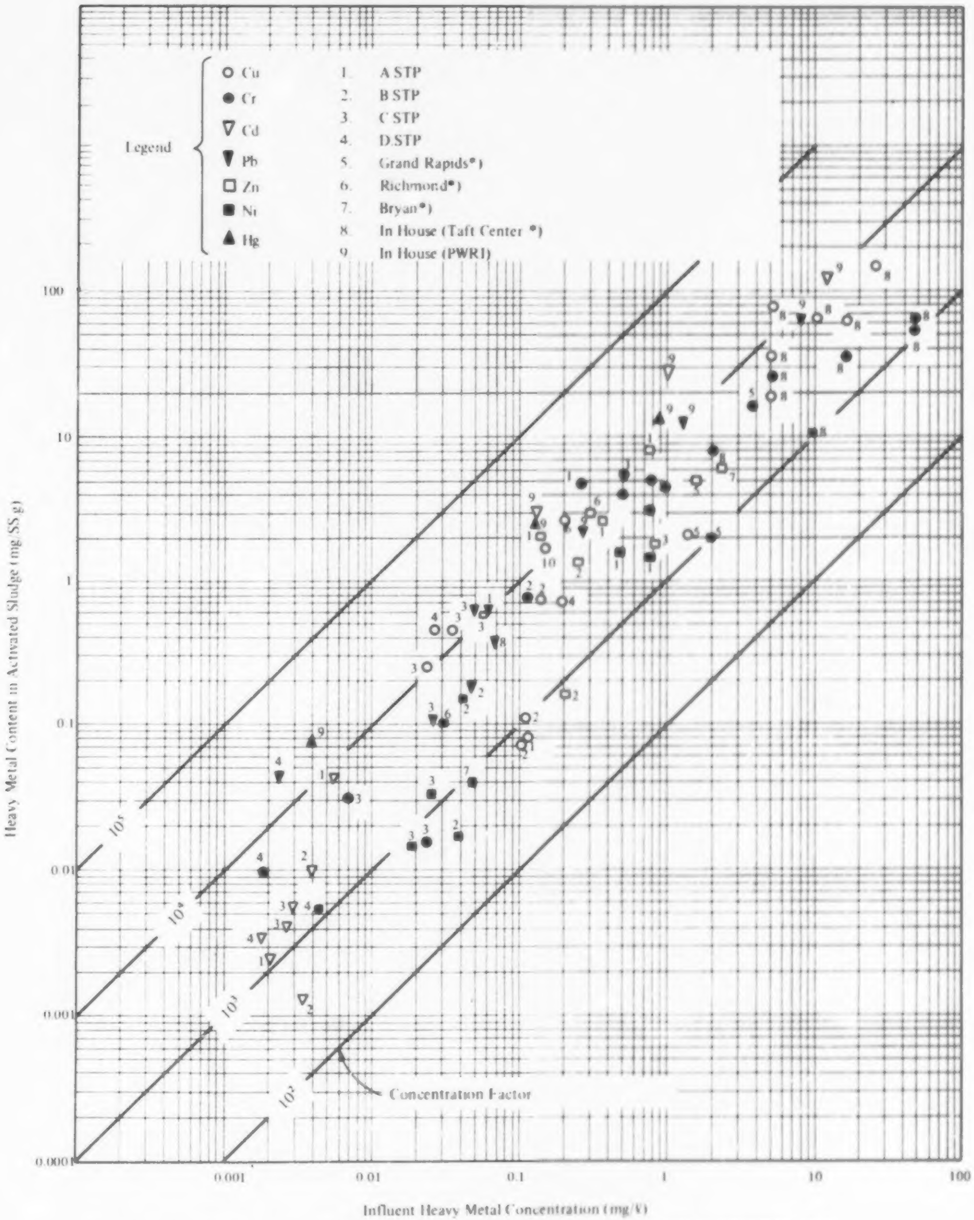


Fig. 1.6 Relationship between Influent Heavy Metal Concentration and Content in Activated Sludge



*) After "Interaction of Heavy Metals and Biological Sewage Treatment Processes" U.S. PHS, No.999-WP-22, 1965.

Fig. 1.7 Relationship between Influent Heavy Metal Concentration and Content in Primary Sludge



*) After "Interaction of Heavy Metals and Biological Sewage Treatment Processes" U.S PHS, No999-WP-22, 1965

Fig. 1.8 Relationship between Influent Heavy Metal Concentration and Content in Activated Sludge

CHAPTER 2
OPERATION OF TORIHAMA INDUSTRIAL WASTEWATER PRETREATMENT PLANT IN YOKOHAMA

CONTENTS

Introduction 452
2.1. Outline of Industrial Wastewater Collective Pretreatment Facilities in Torihama District 452
    2.1.1. Features 452
    2.1.2. Flow Sheets of Treatment Methods 452
2.2. Plant Operation 454
    2.2.1. The Number of Advanced Enterprises and the Volume of Influent . . . 454
    2.2.2. Results of Treatment 456
2.3. Operation and Maintenance Cost 459
2.4. Problems and Countermeasures 461
    2.4.1. Problems on Management 461
    2.4.2. Technical Problems 461
    2.4.3. The Volume of Sludge Produced and Their Disposal 462
Conclusion 465

Introduction

The outline of the facilities and treatment results obtained at the Torihama Industrial Wastewater Pretreatment Plants were reported at the Third US/Japan Conference on Sewage Treatment Technology as an example of "Joint Treatment of Industrial and Domestic Wastewater".

Some improvements have been made on the facilities and operation at these plants have been generally satisfactory from that time on. So we would like to analyze the operational data obtained along with plant operation/maintenance cost and technical problems to be solved.

2.1. OUTLINE OF INDUSTRIAL WASTEWATER COLLECTIVE PRETREATMENT FACILITIES IN TORIHAMA DISTRICT

"Control over the sources" is the basic rule to follow and the same holds true with industrial wastewater we have two approaches to choose from: separate treatment at individual factories and collective treatment as a practical means to take. Considering the fact that, at Torihama industrial district, some 90 percent of the firms advanced into this industrial district are medium and small enterprises with employees of 100 or less, we have adopted, rather than independent individual treatment wastewater, joint pretreatment approach, to admit their wastewater into public sewarage.

Major features and treatment methods are as follows:

2.1.1. FEATURES

(1) The construction and maintenance costs relating to the joint treatment of industrial wastewater is a full charge to constituent enterprises, and the construction and operation/maintenance activities are placed under the control of the City.

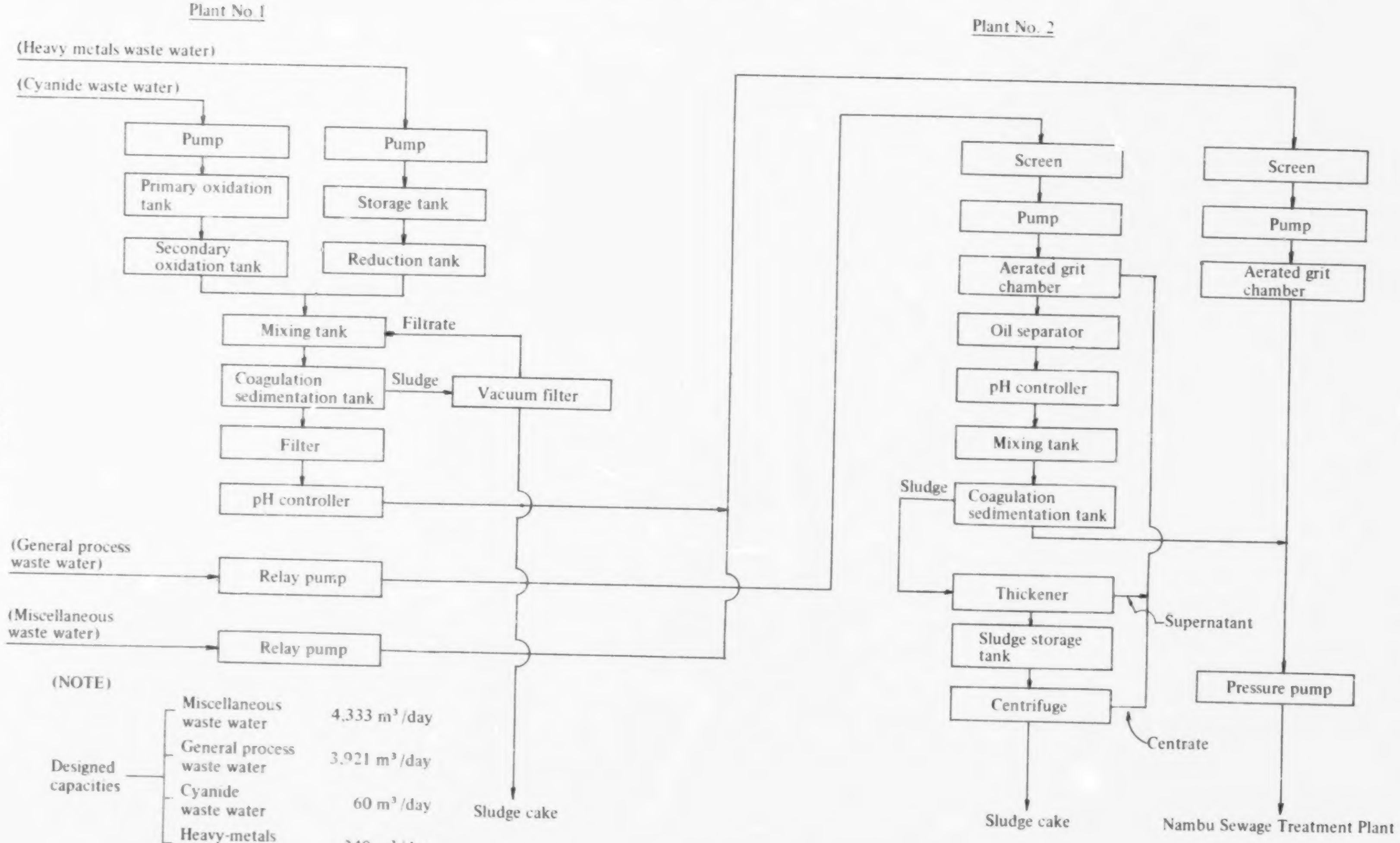
(2) Industrial wastewater is classified into three types: miscellaneous wastewater (from water closets, kitchens, etc.), general process wastewater (containing organic matter, oils, etc.), and pickling-plating process wastewater (discharged from pickling-plating factories). Pickling-plating process wastewater is further divided into two types, that containing cyanide and that contianing heavy metals. Each type of wastewater is led to the treatment plant through separate piping and treated properly according to its particular physical and chemical properties.

(3) Loans of comparatively low interest are being offered by the City to medium and small firms as a public nuisance prevention fund to help them finance their shares of the joint wastewater pretreatment plant construction costs.

2.1.2. FLOW SHEETS OF TREATMENT METHODS

Flow sheets of the treatment methods are shown in Fig. 2–1.

Fig. 2–1 Flow-sheet showing treatment processes at Torihama Industrial Waste Water Pretreatment Plant


Plant No.1
(Heavy metals waste water)
(Cyanide waste water)
Pump
Primary oxidation tank
Secondary oxidation tank
Pump
Storage tank
Reduction tank
Mixing tank
Filtrate
Coagulation sedimentation tank
Sludge
Vacuum filter
Filter
pH controller
(General process waste water)
Relay pump
(Miscellaneous waste water)
Relay pump
Sludge cake
Plant No. 2
Screen
Pump
Aerated grit chamber
Oil separator
pH controller
Mixing tank
Sludge
Coagulation sedimentation tank
Thickener
Supernatant
Sludge storage tank
Centrifuge
Centrate
Sludge cake
Screen
Pump
Aerated grit chamber
Pressure pump
Nambu Sewage Treatment Plant

(NOTE)

Designed capacities	Miscellaneous waste water	4,333 m^3/day
	General process waste water	3,921 m^3/day
	Cyanide waste water	60 m^3/day
	Heavy-metals waste water	340 m^3/day
Plant area	Plant No. 1	1,097 m^2
	Plant No. 2	3,300 m^2

2.2. PLANT OPERATION

The Torihama Pretreatment Plant No. 2 has been in operation since April of 1972, and the Torihama Pretreatment Plant No. 1 since March of 1973. During the two or three years of their operation, later changes have been incorporated in the facilities to improve capabilities of the facilities, and their operating performance is generally satisfactory from that time on.

Servicing status from the beginning of plant operation until now is as follows:

2.2.1. THE NUMBER OF ADVANCED ENTERPRISES AND THE VOLUME OF INFLUENT

The year-by-year increase in the number of constituent enterprises (firms advanced into the industrial district) and the changes in the volume of influent during the period from the end of March, 1973, to the end of March, 1975, are shown in Table 2–1.

Although the advance of enterpirses into the industrial district was initially expected to end by the summer of 1974, the depression by so-called "oil shock" starting in late 1973 has been holding back the pace of advancement. And as of the end of March, 1975, the advance rate (%) was approximately 76% with respect to the object level, while the volume of influent was only 22% against the level originally planned, a fact far behind the object level.

Table 2–1 The number of advanced enterprises and the volume of wastewater

Items / Type of wastewater	Enterprises					Wastewater				
	Planned	End of March 1973	End of March 1974	End of March 1975	Advanced rate in end of March 1975	Planned	End of March 1973	End of March 1974	End of March 1974	Rate in end of March 1975
Miscellaneous	169	55	82	128	76 (%)	4,333 m^3/day	511 m^3/day	1,064 m^3/day	1,121 m^3/day	26 (%)
General process	108	15	31	65	60	3,921	317	990	709	18
Pickling and plating process	6	–	4	5	83	400	–	157	104	26
					Total	8,654	828	2,211	1,934	22

Particularly, the depression became apparent in every sector of the industry, and enterprise operators' efforts concentrated on saving water consumption through shorter-time operation and (or) improvements in processes resulted a decreased drainage.

2.2.2. RESULTS OF TREATMENT

The qualities of influent and clarified water by type of sewage over the period from January, 1975, to the end of May, 1975, are generally satisfactory and are shown in Tables 2–2 and 2–3, respectively.

Table 2-2 Results of Treatment at Plant No. 1 (Pickling/plating process wastewater)

Date	Type of wastewater	Hue	Odor	Water Temp. (°C)	PH	CN (mg/l)	T-Cr (mg/l)	Cr+6 (mg/l)	S-Fe (mg/l)	Ni (mg/l)	Cu (mg/l)	Zn (mg/l)	Pb (mg/l)	Cd (mg/l)
1975 1. 8	Cyanide	Colorless	None	9	–	–	–	Trace	–	–	–	–	–	
	Metals	Yellow	Cresol	10	2.3	10	7.7	54	37	7.2	9.8	6.7	2.9	Trace
	Effluent	Colorless	Slightly cresol	8	6.2	0.04	Trace	Trace	0.36	Trace	3.1	1.1	Trace	Trace
1.22	Cyanide	Light yellow	None	9	11.3	220	1.4	Trace	9.2	1.2	12	140	0.49	Trace
	Metals	Yellow	None	9	2.4	10	83	53	21	16	11	76	0.56	Trace
	Effluent	Light green	None	8	8.1	1.4	1.8	0.23	0.46	0.39	5.3	0.66	0.24	Trace
2. 5	Cyanide	Colorless	Slightly phenol	10	12.8	230	2.0	Trace	3.7	0.71	11	160	Trace	Trace
	Metals	Light green	Slightly phenol	10	1.1	10	4.0	Trace	2,200	0.71	7.2	180	0.43	Trace
	Effluent	Colorless	Slightly phenol	8	7.8	0.03	Trace	Trace	Trace	0.31	4.7	1.2	Trace	Trace
2.20	Cyanide	Colorless	None	8	12.1	350	–	Trace	–	–	–	–	–	–
	Metals	Yellow	Slightly phenol	9	1.6	2.5	–	80	–	–	–	–	–	–
	Effuluent	Light green	Slightly phenol	7	6.8	0.37	–	Trace	–	–	–	–	–	–
3. 5	Cyanide	Colorless	Slightly phenol	10	12.0	460	0.17	Trace	4.8	0.42	6.8	330	Trace	Trace
	Metals	Yellow	Slightly phenol	11	3.6	2.0	55	31	26	6.7	4.7	51	0.21	Trace
	Effluent	Light yellow	Slightly phenol	10	8.9	1.0	0.11	Trace	Trace	0.54	8.0	1.0	Trace	Trace
3.19	Cyanide	Colorless	None	10	12.0	300	–	Trace	–	–	–	–	–	–
	Metals	Yellow	Slightly phenol	11	2.1	3.7	–	120	–	–	–	–	–	–
	Effluent	Colorless	Slightly phenol	17	7.4	Trace	–	Trace	–	–	–	–	–	–
4. 2	Cyanide	Colorless	None	12	11.3	140	1.2	Trace	1.3	0.79	7.4	94	Trace	Trace
	Metals	Green	None	13	1.8	1.0	92	Trace	1,400	7.8	28	210	Trace	Trace
	Effluent	Colorless	None	12	8.0	0.15	Trace	Trace	Trace	0.53	7.3	1.1	Trace	Trace
4.30	Cyanide	Colorless	None	18	11.6	92	Trace	Trace	1.9	20	8.0	47	Trace	Trace
	Metals	Yellow	Slightly phenol	18	1.3	0.67	92	49	46	10	16	160	1.3	Trace
	Effluent	Light yellow	Slightly phenol	18	8.3	–	1.0	0.5	Trace	0.33	0.90	1.1	Trace	Trace
5.14	Cyanide	Colorless	None	19	10.7	130	–	Trace	–	–	–	–	–	–
	Metals	Yellow	None	19	2.2	0.93	–	320	–	–	–	–	–	–
	Effluent	Light yellow	None	20	9.0	0.03	–	0.26	–	–	–	–	–	–
5.28	Cyanide	Colorless	Mineral oil	19	12.6	250	1.5	Trace	3.2	0.60	3.2	250	Trace	Trace
	Metals	Yellow	None	20	2.2	2.6	69	41	16	7.3	7.6	100	0.54	Trace
	Effluent	Colorless	Slightly phenol	21	7.6	0.04	0.67	Trace	Trace	0.32	2.5	2.4	Trace	Trace
Desired level					5–9	1 or less	2 or less	0.5 or less	10 or less	1 or less	3 or less	5 or less	1 or less	0.1 or less

Table 2–3 Results of Treatment at Plant No. 2 (General process wastewater)

Items Date	Type	Hue	Odor	Water Temp. (°C)	PH	BOD (mg/1)	COD (mg/1)	SS (mg/1)	Oil (mg/1)
1975 1. 8	Influent	Turbid black	Meneral oil	9.0	7.0	810	370	580	210
	Effluent	Light green	Hydrogen sulfide	9.0	7.1	140	100	160	43
2. 5	Influent	Turbid grey	Mineral oil	10.0	7.6	240	250	110	46
	Effluent	Light green	Sewage	10.0	8.5	84	100	39	35
3. 5	Influent	Red brown	Sewage	10.0	8.8	580	220	140	32
	Effluent	Light green	Sewage	10.0	8.3	250	110	44	21
4. 2	Influent	Red violet	Sewage	12.0	7.2	320	130	240	120
	Effluent	Red violet	Sewage	13.0	6.8	150	110	10	26
5.28	Influent	Black	Sewage	19.0	7.5	301	210	100	38
	Effluent	Dark green	Hydrogen sulfide	19.0	7.4	311	210	10	16
Desired level		–	–	–	5–9	300 or less	–	300 or less	35 or less

Unsatisfactory copper treatment is supposedly due to insufficient oxidation-decomposition of copper cyanide, because copper in the cyanide wastewater forms complex cyanide and as a result, copper still remains in the effluent. You may also notice that hexa-valent chromium is found in the cyanide wastewater or cyanide is detected in the heavy metals wastewater. This is generally attributable to, inspite of the fact that sewage pipings in individual factories are so arranged as to separate wastewater by its type, incomplete isolation or misoperation by factory operators. To correct this condition which presents difficulties in proper treatment processes, we have been keeping in touch with factory operators and providing them with necessary information by holding guidance sessions for improved process methods and for more strict plant management.

Treatment of general process wastewater at Plant No. 2 is generally satisfactory with minimum variations in influent qualities in recent years.

2.3. OPERATION AND MAINTENANCE COSTS

Operation and maintenance costs including chemicals, lighting and heating expenses, power, personnel, sludge (waste oil) disposal and pipe cleaning expenses are totally charged to constituent enterprises. Allotment of these expenses is based upon the quantities and qualities of wastewater discharged from individual firms.

Shown in Table 2–4 are the quantities of water treated by type of wastewater, operation/maintenance cost, and treatment cost per one cubic meter of wastewater in 1974, as contrasted to those in 1973.

In the 1974 operation cost, because the inflation initiated by the late 1973 energy crises pushed up such expenses as power, personnel, and chemicals 30 to 100 percent, the unit cost of pickling-plating process wastewater treatment increased to a very high level.

Table 2–4 The volume and operation/maintenance cost of wastewater

Type of wastewater	1973			1974		
	Processed volume	Operation/maintenance cost	Average cost per 1 m^3	Processed volume	Operation/maintenance cost	Average cost per 1 m^3
Miscellaneous	264,338 m^3/year	3,774,000 yen/year	14.3 yen/m^3	379,116 m^3/year	8,356,000 yen/year	22.0 yen/m^3
General process	179,224	10,956,000	61.0	272,433	14,605,000	53.6
Pickling/plating process	73,096	15,386,000	210.5	45,571	15,113,000	331.0
Total	516,658	30,116,000		697,120	38,074,000	

As for pickling-plating process wastewater in particular, unusual increase in it is responsible for a substantial drop in the volume of discharged drainage, as it was described earlier, because of the depression.

2.4. PROBLEMS AND COUNTERMEASURES

2.4.1. PROBLEMS ON MANAGEMENT

(1) Treatment Capacities of Collective Pretreatment Facilities

The Processing capacity of this plant was decided on the basis of estimates provided in reports from individual enterprises as to the quantities and qualities of wastewater before the plant was constructed and placed into service. As a result, it is difficult for the plant to allow constituent enterprises to increase their drainage volumes or change the qualities of their wastewater along with development and expansion of their business activities.

In view of our experiences with the plant, therefore, it is desirable, to design a plant with flexibility in its capabilities to some extent to accommodate future expansion, when a new collective pretreatment facility construction plan, will be made.

(2) Official Determination of Wastewater Qualities

As for the determination of quality of wastewater, individual enterprise operators are required to submit a report on it. We found however that these stated qualities in many cases differ greatly from the fact and lack reliability. Therefore, it is desirable that measurements be made by city officials.

(3) Expense Allotment of Operation/Maintenance Cost

Operation and maintenance costs are computed on a liquidation principle at this plant, and this system on a year-by-year expense allotment basis has experienced such a shortcoming: highly varying unit cost of treatment in consequence of such factors as decreased drainage resulting from a depression and soaring management costs by inflation, could be quite a burden to enterprise in carrying their business activities.

For this reason, it is desirable that the present system should be replaced with a new expense allotment system based on revising service fees every two or three years service fees offered at regular rates for two or three years.

(4) Nonfulfillment of Management Contract (delayed allotment payment, nonperformance of water quality reporting obligation, etc.)

Because of the fact that the City takes primary responsibility for plant operation and maintenance, there appears a tendency that enterprise operators are less conscious about their responsibility than before, and in fact, we have difficulty in obtaining cooperation from some of the operators with nonperformance of management contracts, etc.

2.4.2. TECHNICAL PROBLEMS

(1) Treatment of Wastewater Containing Oils and Fats

At this collective pretreatment plant, waste oils and fats isolated and removed by aeration system. However, isolation performance is not satisfactory because of the fact that various kinds of oils and fats are admitted to the facilities. To adopt a treatment method best suited for the properties of oils in a free or emulsified state, therefore, we believe that an individual treatment is more efficient than a collective treatment.

(2) Pickling-Plating process Wastewater Treatment

Pickling-plating wastewater was divided into two types, that containing cyanides and that containing heavy metals. But further seperation of the latter was impossible for reasons of complexity and technical difficulties of piping, so that chromium wastewater and acid/alkaline wastewater were allowed to flow in one piping system. As a result, we faced the following problems:

1) Treatment of heavy metals is as follows: the hexa-valent chromium is reduced to trivalent chromium first by lowering its pH value and then, by increasing its pH value, it is separated together with other metals, and removed as hydroxides. But this process changes not only the pH value of the chromium wastewater, but also that of the acid/alkaline wastewater, and as a result the consumption of pH adjusting agents such as sulfuric acid and calcium hydroxide increased. It also meant extra sludge production and added operation cost.

2) Hexa-valent chromium reduction is conducted by automatically controlling the feed rate of chemicals in proportion to the pH value and oxidation-reduction potential. But because the oxidation-reduction potential is affected by iron ions, contained substantially in the alkaline wastewater, to compensate for this condition a reductant is added slightly more than that normally required.

3) It is a generally suggested rule that concentrated wastewater produced from plating factories be pooled temporarily within the factories and then drained in small quantities so that it is sufficiently diluted with usual wastewater. However, diluting operation by individual firms isn't always performed satisfactorily.

2.4.3. THE VOLUME OF SLUDGE PRODUCED AND THEIR DISPOSAL

(1) Plant No. 1

The 1974 plant operation records show that sludge cakes (moisture content 80%) averaged 880 kilograms a day in weight.

Because pickling-plating process wastewater is treated in this plant, sludge cakes contain such materials as heavy metals It is necessary, therefore, that some means be employed so that they would not ooze out of sludge cakes upon disposal. The means applicable we have experimented or studied so far are (1) solidification with concrete, (2) sintering, (3) melting, (4) reduction sintering, and (5) colliery reuse (bringing them back to refineries as raw materials). As a result, considering such factors as feasibility and prevention of further leaching, sintering method is under consideration as the one of practical approaches today. As for this method, a flow sheet along with part of its experimental results was summarized in the previous report. And this time, it is presented in the following Table and Figures together with its possible application to synthetic aggregate.

Table 2–5 Physical characteristic of sintered sludge

Sintering additives	Blending ratio Plating sludge : Clay	Sintering temperature (°C)	Sintering time (Minute)	Granulating method	The state of sintered sludge	Strength of crushing (kg)		Specific gravity	Water absorb (%)
						Average	Min.–Max.		
Clay	1 : 1	1,100	60	By machine	Half fused, Black-brown	39	35– 47	–	–
Clay	1 : 2	1,100	60	By machine	Sinter, Black-brown	52	44– 69	–	–
Clay	1 : 3	1,100	60	By machine	Sinter, Brown	25	20– 30	–	–
Clay	1 : 4	1,100	60	By machine	Sinter, Light red-brown	25	20– 32	2.1	2.7
Clay	1 : 4	1,150	60	By machine	Sinter, Brown	35	20– 44	1.7	3.0
Clay	1 : 4	1,200	60	By machine	Sinter, Brown	59	49– 69	1.6	0.03
Clay	1 : 5	1,200	60	By hand	Sinter, Brown	93	86– 98	1.8	0.82
Clay	1 : 6	1,200	60	By hand	Sinter, Brown	105	93–113	2.1	0.67
Refractory material	1 : 3	1,100	60	By machine	Half sinter, Black-brown	15	15– 15	2.1	12.9
Refractory material	1 : 3	1,150	60	By machine	Sinter, Black-brown	45	44– 47	2.2	0.84
Refractory material	1 : 3	1,200	60	By hand	Half fused, Black-brown	96	60–123	2.3	0.47
Grit-stone	–	–	–	–		57	44– 69	–	–

Fig. 2–2 Solubility test of various PH in elutriate

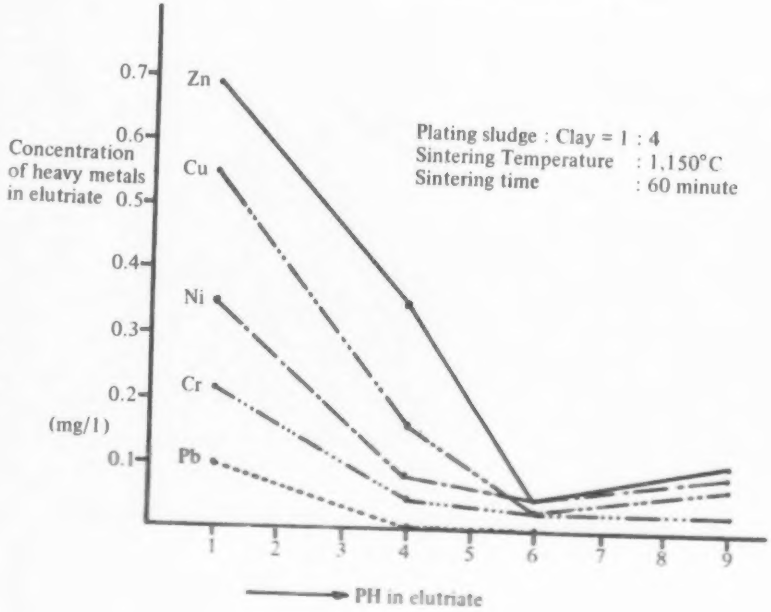
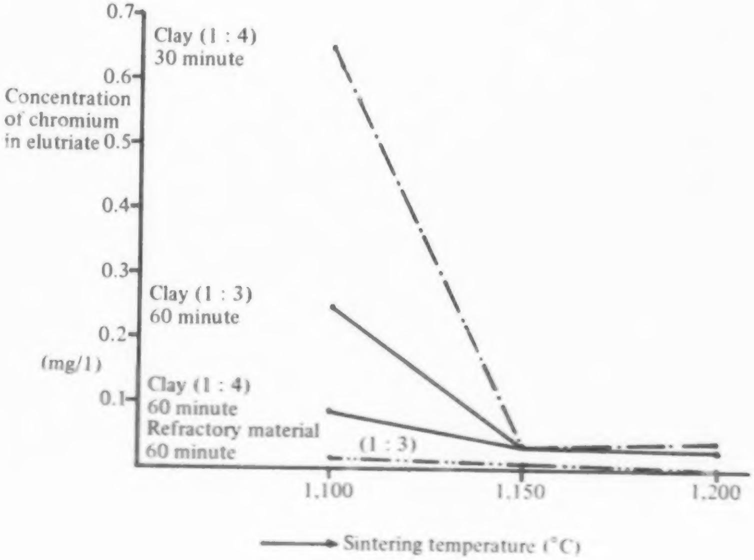


Fig. 2–3 Sintering temperature and solubility of chromium



(2) Plant No. 2

The 1974 operational records show that sludge cakes (moisture content 80%) averaged 435 kilograms a day. Because their composition is much the same as that of domestic sewage, they are disposed as land fill, together with sludge produced from other sewage treatment plants.

Conclusion

The example we presented in this report is a counter measure against industrial wastewater which has been planned and practiced on the premise that industrial wastewater from medium and small enterprise districts are allowed to flow into public sewarage. We believe that this approach is one of the most efficient ways, at least for the time being, and acceptable by to small and medium enterprises that can hardly afford, technically and financially, an independent treatment.

We are generally satisfied with the operational results obtained, however, (1) enterprise operators' sense of responsibility for their fulfillment of obligations, (2) with the plant's capacity being inflexibly fixed, it is incapable of meeting quantitative and qualitative changes of wastewater resulting from variations in business activities of constituent enterprises, (3) final disposition of sludge, and (4) indiscriminately mixed up inflow of various wastewaters are among the problems we must solve urgently.

CHAPTER 3. FUKASHIBA INDUSTRIAL WASTEWATER TREATMENT PLANT IN IBARAGI PREFECTURE

3.1 General Plant Review 467
3.2 Quality and Quantity of Industry Wastewater. 467
3.3 Charge. 469
3.4 General Condition of Maintenance and Operation. 469
3.5 The Present Sludge Disposal 469
3.6 Problems. 469

3. FUKASHIBA INDUSTRIAL WASTEWATER TREATMENT PLANT IN IBARAGI PREFECTURE

The description of this plant was informed at the first U.S.-Japan Conference Sewage Treatment Technology in 1971 and the general survey of the maintenance and operation of the plant from Sept., 1971 to June, 1975 is given below.

3.1 GENERAL PLANT REVIEW

This treatment plant was designed to provide preliminary and secondary treatment to the wastewater of the oil refineries and the petrochemicals existing in the area of 1,800 ha except steel mills area in the Kashima industrial district of 2,400 ha and the future wastewater flow is estimated about 300,000 m^3/day.

The preliminary treatment facilities include grit chambers, oil floatation tanks, pH control tanks and the chemical coagulation and settling tanks by which B.O.D. load for subsequent biological process is reduced and the hazardous substances for metabolism are removed as much as possible.

The secondary treatment facilities include aeration tanks and final setting tanks, operating as a conventional activated-sludge system for purpose of the stabilization of final effluents. The secondary facilities also may be operated as a two stage activated-sludge system if necessary, in case of the striking fluctuation of influent quality.

In order to prevent the transpiration of odor arising from plant site, Oil floatation tanks, pH-control tanks and sludge thickeners are covered with lids and the odor is removed by a soil adsorption process or an activated carbon absorption process.

Sludge is mechanically dewatered after thickening and then to be incinerated. (Fig. 1)

3.2 QUALITY AND QUANTITY OF INDUSTRY WASTEWATER

Effluent of Fukashiba Wastewater Treatment Plant is discharged into the sea of Kashima and the effluent standards are established by Ibaragi Prefectural Regulations as shown in Table 1.

In order to meet the regulations, it is required that the strength of influent into the treatment plant should not exceed approximately 200 mg/ℓ of COD, 250 mg/ℓ of suspended solid and 10 mg/ℓ of Hexane Solubles.

Therefore, each industry is demanded annually to report quantity and quality of his wastewater to administrative office. The data are adjusted and effluent criteria for each industry (agreement rules) are determined (Table 2).

Most of industries are required pre-treatment facilities to meet the criteria. The existing pre-treatment facilities are as shown in Table 3.

The kinds of the facilities are mostly pH control tanks, sedimentation tanks and oil separators. Some factories are required special treatment equipments such as stripping towers to remove volatile organic matters and spray incineration equipments to treat high-COD wastewater.

REFERENCE

S. Matui, Activated Sludge Degradability of Organic Substances in the Wastewater of the Kashima Petroleum and Petrochemical. Industrial Complex in Japan, 7th International Conference on Water Pollution Research, Paris, Sept. 9 ~ 13, 1974.

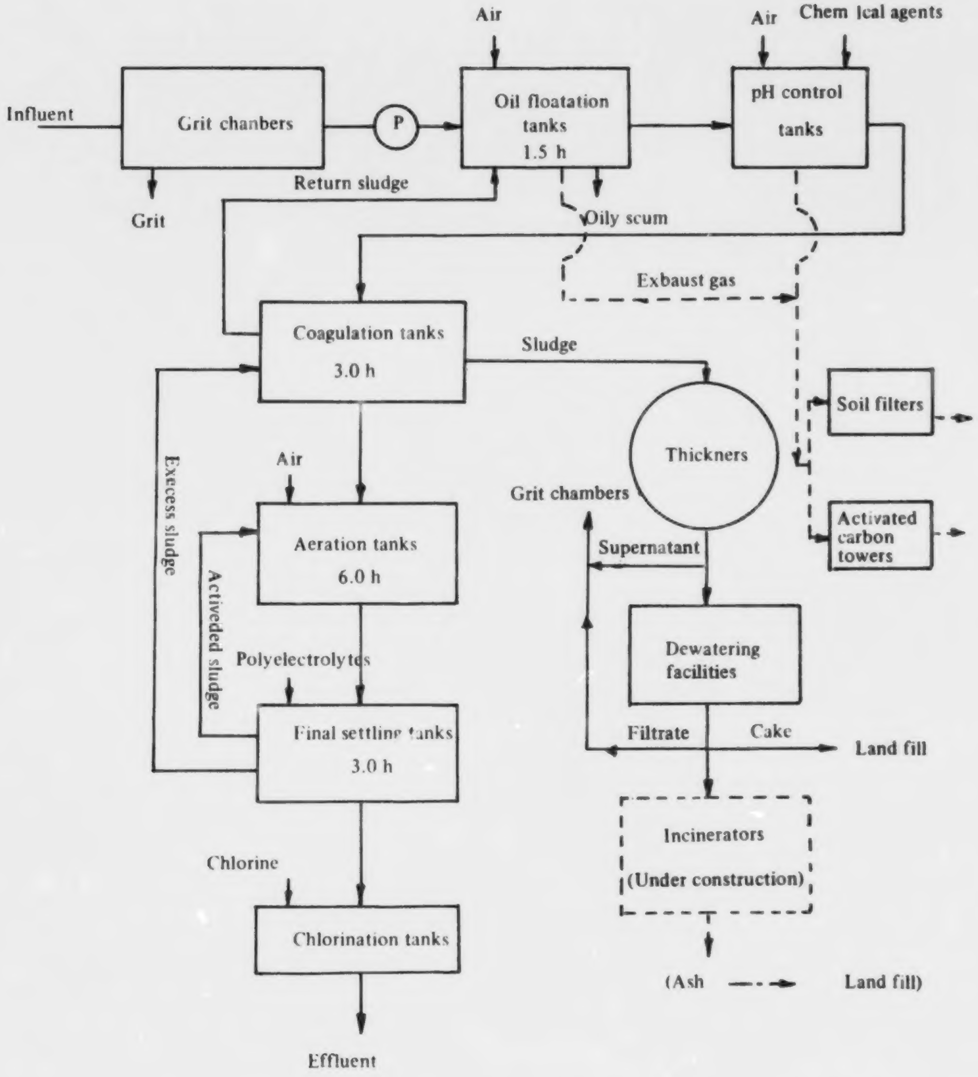

Air
Air
Chem ical agents
Influent
Grit chanbers
P
Oil floatation tanks 1.5 h
pH control tanks
Return sludge
Grit
Oily scum
Exbaust gas
Coagulation tanks 3.0 h
Sludge
Soil filters
Thickners
Execess sludge
Air
Grit chambers
Activated carbon towers
Aeration tanks 6.0 h
Supernatant
Activeded sludge
Polyelectrolytes
Dewatering facilities
Final settling tanks 3.0 h
Filtrate
Cake
Land fill
Chlorine
Incinerators (Under construction)
Chlorination tanks
(Ash
Land fill)
Effluent

Fig. 1 Process Flow of Fukashiba Wastewater Treatment Plant

3.3 CHARGES

Quantity and Quality Charge System is applied for all industries discharging into the sewers

Namely, below the agreement rules, the charge is calculated by using the Quantity and Quality Charge formula. If the quantity and/or quality of wastewater from any factory exceed the agreement, surcharge is levied to the wastewater below the effluent standards of Sewerage law. And exceeding the effluent standards of the Sewerage law, penalty is imposed. (Table 4)

Each factory is required to install flow meters, partial type or electromagnetic type in order to measure the wastewater quantity which is the basis of the assessment of the charge. Each factory also is required to install automatic pH recorders, Toc meters etc. In addition, the continuous monitoring and the occasional quality examinations of once to four times a month are performed to secure the effect.

3.4 GENERAL CONDITION OF MAINTENANCE AND OPERATION

The data of operations of this treatment plant in each month from January to June, 1975 are as shown in Table 5.

Most of industries keep operating continuously around the clock and the fluctuation of quantity and quality is little. However, the strength of chlorine ion is approximately 3,500 mg/ℓ and seems to be higher than in influent into other treatment plants.

The removal rate of suspended solids is high in chemical sedimentation tanks and 80%, but those of BOD, COD and Haxane Solubles are low and 38, 15 and 33%, respectively.

And the removal rates of them are high in final settling tanks except Haxane Solubles and B.O.D., C.O.D. and Suspended Solids are 90, 70 and 50% removal, respectively. But the removal rate of Haxane Solubles is 25% and it is as almost same as in chemical sedimentation tanks. (See Table 4)

Besides, 1 mg/ℓ of polyelectrolytes such as polyethylene aluminium chloride is added in final settling tanks to promote sedimentation of sludge flocs.

3.5 THE PRESENT SLUDGE DISPOSAL

Sludge is drawn from chemical sedimentation tanks and concentrated. Dewatering is done by Filter Press Method on a dosage of about 19 to 31% slaked lime of dry solids. Sludge cakes are disposed by land fill, but incinerators are now under construction and they are to be completed in Dec., 1975.

And these sludge cakes are planned to be utilized as soil enrichment agents and fertilizer for plants in the sandy soil and the pot studies and the field plot examinations of 3,000 m^2 farm are provided.

The results of analysis of heavy metals contained in the sludge cakes and the results of elution tests are as shown in Table 6 and 7 and there are no problems.

3.6 PROBLEMS

1. In the annual routine maintenance of industries BOD load and chlorine ion are strikingly reduced, the operating control is difficult and the effluent quality sometimes grow worse.
2. As the industrial wastewater sometimes contains high reducing materials, it is necessary that Dissolved Oxygen in the aeration tank, is kept high level.

Table 1 Effluent Standards for the Sea of Kashima

Constituent		Concentration (mg/ℓ)	
		1971	1975
1.	Cadmium (Total)	0.1	0.1
2.	Cyanide	1	1
3.	Organic phosphide	1	1
4.	Lead (Total)	1	1
5.	Chromium (Total hexavalent)	0.5	0.5
6.	Arsenic (Total)	0.5	0.5
7.	Mercury (Total)	ND	0.005
8.	Alkyl mercury compounds	ND	0.005
9.	pH	5.0 ~ 9.0	5.8 ~ 8.6
10.	C.O.D.	120 (100)*	50 (40)*
11.	Suspended solid	60 (50)*	50 (40)*
12.	Fecal coliforms		3,000
13.	Hexane solubles (Mineral oils)	7 (5)*	3 (2)*
	(Fats and fatty oils)	7 (5)*	3 (2)*
14.	Phenols	5	5
15.	Copper (Total)	3	3
16.	Zinc (Total)	5	5
17.	Iron (Dissolved)	10	10
18.	Manganese (Dissolved)	10	10
19.	Chromium (Total)	2	2
20.	Fluoride (Total)	15	5

* Daily average values are shown in parentheses.

Table 2 Agreement Values (1977)

Factories	Discharge	pH	B.O.D	C.O.D.	Suspended solid	Haxane solubles
	m³/day		mg/ℓ	mg/ℓ	mg/ℓ	mg/ℓ
Tobu District						
1	15,500	5 ~ 9	80	80	15	15
2	1,830	5 ~ 9	250	300	80	6
3	35	6 ~ 9	150	150	200	0
4	1,910	5.5 ~ 8.5	249	242	65	7
5	11,000	5 ~ 9	470	300	170	15
6	400	5 ~ 9	90	70	300	3
7	4,700	5 ~ 9	150	300	30	10
8	14,100	5 ~ 9	160	160	40	3
9	2,300	5 ~ 9	27	41	35	8
10	5,000	5 ~ 9	270	200	40	10
11	6,700	5 ~ 9	60	150	65	5
12	250	5 ~ 9	50	150	100	6
13	3,500	5.8 ~ 8.6	100	220	25	8
(14)	2,000	6 ~ 8	250	250	100	3
15	9,500	5 ~ 9	200	300	50	5
16	3,000	6 ~ 8	60	60	50	5
17	1,500	5 ~ 9	30	30	40	10
18	300	6 ~ 9	300	250	50	10
19	100	5 ~ 9	15	100	100	10
20	100	5 ~ 9	350	300	50	20
Seibu District						
(21)	150	6 ~ 8	150	150	200	5
(22)	200	6 ~ 8	50	50	50	1
23	195	6.5 ~ 8.6	220	160	160	10
(24)	3,790	5 ~ 9	300	150	50	10
25	2,000	5 ~ 9	200	80	90	20
26	30	7.0	100	100	100	0
27	9,000	5 ~ 9	300	300	30	10
(28)	1,340	7.5	10	20	20	0.5
29	208	6 ~ 8	100	100	80	7
(30)						
(31)	4,000	6 ~ 8	25	20	50	20
(32)	1,000	5 ~ 9	20	40	20	7
Hazaki District						
(33)	600	5 ~ 6	300	200	50	7
(34)	720	6 ~ 9	250	100	10	5
35	5	5.8 ~ 8.6	22	22	10	5
(36)	180	5.5 ~ 8.5	50	50	20	5
37	200	6 ~ 8	60	10	50	1
(38)	710	5 ~ 9	10	20	50	10
(39)	520	5.7 ~ 8.7	160	100	200	6
(40)	21	5.6 ~ 8.6	90	80	250	0
(41)	126	6.5 ~ 8.0	300	300	40	5
(42)	90	5.5 ~ 8.5	70	140	50	7
(43)	272	6.5 ~ 7.5	100	200	20	5
(44)	1,400	5 ~ 9	300	300	20	10
(45)	640	6 ~ 8	300	300	150	10
(46)	221	6 ~ 8	68	37	18	1
(47)	249	6.5 ~ 8.5	60	40	90	4
(48)	270	7	22	38	70	4
(49)	810	6.5 ~ 8.5	20	10	15	5
(50)	54	5 ~ 9	5	16	10	0
(51)	1	6 ~ 8	30	30	30	5
Total	112,727	5 ~ 9	186	182	57	9

Future industries are shown in parentheses.

Table 3 Existing Pre-treatment Facilities

Factories	pH control tanks	Oil separators	Sedimentation tanks	Activated sludge facilities	Others
1	○	○	○		
2	○		○		Stripping tower
3					
4	○		○		
5	○	○	○	○	
6	○				
7	○				
8	○		○		
9			○		
10	○			○	
11	○			○	
12			○		
13		○	○		
15	○		○		
16	○		○		
17	○	○			Stripping tower
18	○	○			
19	○	○			
20	○				
25		○	○		
27	○		○		Spray incineration equipment
35	○				
Total	17	7	12	3	

Table 4 Industrial Wastewater Charge System of Fukashiba Wastewater Treatment Plant (Effective April 1, 1975)

Charge = quantity charges + quality charges + surcharges + penalties

1. **Quantity charges**

32 yen per m^3 per a day of quantity (Qe) (10 yen in 1971)

Where;

Quantity (Qe) (m^3/day) = Q (below of 3,000 m^3/day)

or

$= 3{,}000 + 0.9\,(Q - 3{,}000)$

(exceed Q of 3,000 m^3/day and below Q of 5,000 m^3/day)

or

$= 4{,}800 + 0.8\,(Q - 5{,}000)$

(exceed Q of 5,000 m^3/day)

Q is a daily average flow (m^3/day) which is measured by a flow meter.

2. **Quality charges**

Quality charge per m^3 per a day of "Q" is as follows:

Strength (F)	Charges 1971	 1975
	Yen	Yen
$F < 120$	4	20
$120 \leqq F < 240$	6	30
$240 \leqq F < 360$	8	40
$360 \leqq F < 480$	10	50
$480 \leqq F < 600$	12	60
$600 \leqq F < 720$	14	70
$720 \leqq F < 840$	16	80
$840 \leqq F < 960$	18	90
$960 \leqq F < 1080$	20	100
$1080 \leqq F < 1200$	22	110
$1200 \leqq F < 1320$	24	120

Where;

$$F = \frac{B + C}{2} + S + 6N$$

in which B, C, S and N are BOD_5, COD, SS and Hexane Solubles in mg/ℓ which are determined due to analize samples, respectively.

3. Surcharges

(1) When quantity of "Q" exceeds 110% of the agreement values, 64 yen is payed for each m^3 per a day of excess quantity over the agreement values.

(2) When the strength of the wastewater exceeds 120% of the agreement values, the following charges are payed for each m^3/day of quantity of "Q".

Surcharge (yen/m^3/day)		
1971	1972	
3	12	When "F" based on the determined strength is below "F" based on the agreement values or exceeds one step over it.
6	24	When "F" based on the determined strength exceed two steps over it.
10	40	When "F" based on the determined strength exceeds three steps over it.

4. Penalties

When the determined strengths of any constituents of wastewater exceed the following standards, penalties of 40 yen (10 yen in 1971) per m^3 per a day of quantity is imposed for each item of violation.

Items	Criteria
Temperature	45°C
pH	$5 \leqq pH \leqq 9$
BOD_5	600 mg/ℓ
COD	600 mg/ℓ
Suspended solids	600 mg/ℓ
Hexane solubles	20
Others	Those required by the administrator.

Table 5 Monthly Operation Data of Fukashiba Wastewater Treatment Plant

Month	Influent								Aerated oil separators and pH control tanks	Chemical coagulation tanks				
	Quantity (m³/day)	Temperature (°C)	pH	BOD (mg/ℓ)	COD (mg/ℓ)	Suspended solids (mg/ℓ)	Hexane solubles (mg/ℓ)	Chlorine ion (mg/ℓ)	Cu.M. air per Cu.M. wastewater	Detention time (hr)	BOD (mg/ℓ)	COD (mg/ℓ)	Suspended solids (mg/ℓ)	Hexane solubles (mg/ℓ)
1975 Jan.	42,800	22	7.4	184	144	155	7	2,790	4.6	6.2	82	119	14	3
Feb.	51,800	21	7.6	84	145	190	6	2,330	3.7	3.5	71	128	15	4
Mar.	55,400	22	7.4	154	158	116	6	2,700	3.9	3.0	101	141	33	4
Apr.	56,300	26	7.5	103	132	115	5	4,110	3.0	3.0	64	107	19	4
May	56,500	30	7.5	107	118	72	6	4,390	3.4	3.0	66	99	40	4
June	56,100	29	7.6	106	127	63	5	3,430	3.7	3.7	73	104	10	4
Avg.	53,200	25	7.5	123	137	119	6	3,290	3.7	3.7	76	116	24	4

Month	Aeration tanks							Final settling tanks					Thickener		Dewatered sludge	
	Cu.M. air per Cu. M. wastewater	MLSS (mg/ℓ)	VSS/SS (%)	SVI	MLDO (mg/ℓ)	Excess sludge (Ton/dry solid/day)	Return sludge ratio (%)	BOD (mg/ℓ)	COD (mg/ℓ)	Suspended solids (mg/ℓ)	Hexane solubles (mg/ℓ)	Coagulant (mg/ℓ)	Sludge volume (m³/day)	Water contents (%)	Dosage CaO (%)	Weight of sludge cakes (Ton/dry solid/day)
1975 Jan	10.6	3,750	86.5	51	4.3	0.67	25	7	33	12	2	2.0	115.9	93.1	21.3	10.5
Feb.	8.5	3,870	86.5	46	4.7	0.76	23	6	35	8	3	1.5	188.3	94.2	30.0	15.4
Mar.	9.0	3.970	82.1	46	5.8	0.62	23	6	33	15	3	0.8	213,3	95.2	23.9	13.2
Apr.	7.0	3,250	79.8	46	6.0	0.67	23	3	30	17	3	0.2	185.2	94.2	19.1	13.1
May	8.1	3,710	76.7	47	3.8	1.03	21	7	40	7	3	0.6	161.2	94.9	22.1	10.6
June	8.7	3,350	78.8	48	4.1	1.30	22	10	36	12	3	1.3	181.6	94.1	31.4	14.7
Avg.	8.7	3,650	81.7	47	4.7	0.84	23	7	35	12	3	1.1	174.3	94.2	23.6	13.1

Table 6 Heavy Metals Contained in Sludge Cakes

(mg/kg of dry solids)

Month	1975					
	Jan.	Feb.	Mar.	Apr.	May	June
Cadmium	4.0	5.2	6.4	8.7	7.8	8.8
Lead	220	17	38	133	25	92
Chromium	1,048	770	920	1,320	4,640	1,300
Iron	880	324	1,100	960	1,420	725
Copper	1,840	1,280	720	780	668	432
Zinc	195	1,104	1,400	2,400	1,440	834
Manganese	264	268	198	296	238	208
Nickel	208	184	180	700	180	100
Arsenic	4.0	7.5	1.6	8.5	2.2	3.6
Mercury	–	–	–	–	–	0.012

Table 7 Strength of Heavy Metals in Supernatant in Elution Tests for Sludge Cakes

Sample*	I	II	III	Criteria
Mercury (Total)	ND	ND	ND	0.005
Alkyl Mercury	ND	ND	ND	0.005
Cadmium (Total)	0.01	ND	ND	0.3
Lead (Total)	0.05	ND	ND	3
Organic Phosphide	ND	ND	ND	1
Chromium (Total Hexavalent)	ND	ND	ND	1.5
Arsenic (Total)	ND	ND	ND	1.5
Cyanide	ND	ND	ND	1
Dry Solid**	42.6	42.6	42.6	–

* Samples were prepared as follows:

Sample I:	10 g of sludge cakes was added to 90 g of water and the range of pH was adjusted 5.8 to 6.3. Sample was mixed 6 hours and filtered with filter paper.
Sample II:	As same as Sample I except pH range of 7.8 to 8.3.
Sample III:	3 g of sludge cakes was added to 97 g of water and the range of pH was adjusted 7.8 to 8.3. Sample was mixed 6 hours and filtered with filter paper.

** Expressed in per cent.

CHAPTER 4. DESIGN AND OPERATION OF DYING WASTE TREATMENT FACILITY IN SABAE CITY, FUKUI PREFECTURE

4.1 Production of Coagulant "Activated Diatom Earth" from Diatomaceous Rich in Clay and Its Features 478

4.2 Planning Design and Operation of Dying Wastes Treatment Facility in Sabae City, Fukui Prefecture 479

4. DESIGN AND OPERATION OF DYING WASTE TREATMENT FACILITY IN SABAE CITY, FUKUI PREFECTURE

4.1 PRODUCTION OF COAGULANT "ACTIVATED DIATOM EARTH" FROM DIATOMACEOUS RICH IN CLAY AND ITS FEATURES

The Noto Peninsula, lying in the north of Honshu, abounds in diatomaceous earth containing clay and vocanic ash. Its estimated amount of reserves is said to be as much as several thousands of millions of tons. (See Gig. 4.1)

The average composition of the diatomaceous earth produced in the Peninsula is SiO_2 average 70%, Al_2O_3 8~11%, Fe_2O_3 4~6%, CaO 0.5~1.4%, NgO 0.2~0.25%, and $Na_2O + K_2O$ 1.5~3%, with the content of SiO_2 considerably lower as compared with ordinary one. For this reason, it has not been used as filter agents so far.

The Ishikawa Prefectural Industrial Laboratory has investigated the commercialization of diatomaceous earth buried in the ground in the Noto Peninsula for the purpose of promoting local industries. Their efforts have turned out a coagulant under the trade name of "Activated Diatom Earth".

The production process of this coagulant is shown in Fig. 4.2. Namely, turned-up diatomaceous earth is dried, pulvulized, added with water and sulfuric acid, baked in a rotary kiln at a temperature of 250 to 350°C and again pulvulized as a final product. By baking after being added with water and sulfuric acid, the earth has its aluminum oxide changed into aluminum sulfate and ferric oxide into ferric sulfate, thus gaining a ability to coagulate suspended and coloidal solids. Its ability combined with the adsorption force of diatomaceous earth, is displayed to the full for coagulation.

The coagulant, "Activated Diatom Earth", finds itself in the treatment of some kinds of dying waste and oil containing waste.

In Japan, the guidelines concerning the industrial effluent limitations have not yet specified that for colour. Effluents from dying mills largely vary in pH and contain a considerably large amount of organic substance. The most problematic among others is their own hue and colour of the waste, which in many cases is serving a main cause of trouble with neighbors and downstream inhabitants and farmers.

As a consequence, the methods of reducing the colour of effluents from dying mills have long been investigated for all that they are not specified in the guidelines. The coagulant, "Activated Diatom Earth", is excellent in colour reduction of effluents from dying mills. Table 4.1 shows colour reduction effects and $KMnO_4$-COD removal in chemical coagulation of various kinds of dye solutions.

According to laboratory tests, the Coagulant is able to treat wastes of dispersion dyes, acid dyes, naphtol dyes and vat dyes to discolour to perfection. Also, it goes a long way with the colour reduction of direct dyes, reaction dyes and base dyes.

As regards waste of sulfide dyes, it has little or no efficacy; rather, ferrous sulfate is more effective as a coagulant. "Activated Diatom Earth" has the optimum pH depending on the kinds of dyestuffs at which its colour reduction performance can be exhinited best.

Fig. 4.3 shows the pH vs. colour reduction relationships for acid dyes (Suminol Leveling Red 3B), base dyes (Aizen Cathilon Red K-GLH), and dispersion dyes (Resoline Blue BR).

Another feature of the Coagulant is notable dewatering effect when applied to sludge, and still more important is the fact that the dewatered cake can be used. Excellent dewatering effect is attributable to diatomaceous earth in the Coagulant. The dewatered cake has been mixed with cement and molded into artificial trees or stones for use at parks and gardens.

Recently, a method of manufacturing filter agents from dewatered cakes has been developed and industrialized. Since the dewatered cakes can be turned into secondary products, the manufacturer of the Coagulant has been carried out business taking over dewatered cakes free of charge at users' treatment plants while suppling the Coagulant "Activated Diatom Earth".

Fig. 4.4 shows a flow sheet explaining the production process of filter agent from sludge cakes.

4.2 PLANNING DESIGN AND OPERATION OF DYING WASTES TREATMENT FACILITY IN SABAE CITY, FUKUI PREFECTURE

Sabae City, in Fukui Pref., developed the Eastern Industrial Estate, and has made efforts to attract manufacturing businesses.

Now there are some dying mills in the Estate, and the water quality of the Kurozu River which is receiving their effluents is hideously bad. The inhabitants of downstream of the Kurozu River lodged strong protests against Sabae Municipal Office, claiming to improve water quality.

Meanwhile, the Office planned to construct sewarage system to cover the entire unban areas and a sewage treatment plant.

But, the plan was thought to take too much time to prevent the water pollution of the Kurozu River. Besides, it was feared that the improvement of the secondary effluent in hue and colour at the sewage treatment plant could hardly be expected if the wastewater of dying mills were accepted with domestic sewage by the public sewers.

To cope with this problem, the Sabae Municipal Office has planned to construct a joint pretreatment facility where dying wastewater are treated by a chemical sedimentation process to remove color and a part of organie substances.

While the effluent from the facility is planned to be discharged into the Kurozu River for the time being, it will be treated again at the municipal sewage treatment plant in future, mixed with domestic sewage. In preparation for this plan, the Public Works Research Institute, Ministry of Construction, took a part, and discussed with engineers of both the Municipal Office and the Dying Industry Cooperative Association several times.

As a result, the aforesaid coagulant, "Activated Diatom Earth", was decided upon as the coagulation agent for the Joint Pretreatment Facility.
The reasons are as follows.

i) Of the dyestuffs used in the mills in the Sabae Eastern Industrial Estate dispersion dyes occupy the majority. Others, including acid dyes, naphtol dyes and reaction dyes, are also used. From basic experiments in the past, it has been corroborated that the coagulant "Activated Diatom Earth" is effective in colour reduction of these dyes.

ii) Although the Activated Diatom Earth was priced at ¥25 per kg at that time, far more expensive than other coagulants, and its dosing rate was as high as 1,000 ppm, the latter was expected to be whittled away sharply if technical investigation has been set up.

iii) The settled sludge can be dewatered to a 70% of water content by a vacuum filtration without filter agent. Fortunately, the manufacturer of the coagulant "Activated Diatom Earth" has commited themselves to take over dewatered sludge for making filter agent.

iv) From the above, it is expected that the total costs, including the cost for treatment and disposal of settled sludge, will become cheaper than with other coagulants (e.g., aluminum salts, ferrous salt, ferric salt etc.). Still another advantage is no need of preparation of the land required for disposal of sludge.

As the coagulant, "Activated Diatom Earth", was picked up for the treatment of waste from the dying mills, various experiments have been carried out to find the way to minimize the dose rate of the coagulant and the way to keep stabilized operation of the pretreatment facility. The results are as follows.

i) The waste from the dying mills are very changeable in its flow and quality from time to time. It is therefore necessary to install an equalizing pond. The capacity of the pond is estimated to be at least four-hour's worth of hourly maximum waste water flow in order to keep stable operation of the joint pretreatment facility.

ii) If the waste of the mills is treated in the chemical sedimentation tank after removal of their surface active agents in a foam separation tank, the dosing rate of the coagulant can be reduced by some 30% compared with the amount required when the waste was directly treated in a chemical sedimentation tank.

iii) Part of the coagulant "Activated Diatom Earth" to be dosed into the defoamed waste can be replaced with an inexpensive metal salt (a mixture of aluminum sulfate and ferric sulfate available on market at a cost of about ¥10 per kg under the trade mark of MICS).
So long as the ratio of the "Activated Diatom Earth" to MICS is 3 : 1 or less, there is no problem in dewatering or in making filter agent either.

iv) The optimal pH value at which the Activated Diatom Earth and MICS are to be reacted upon the waste is in the range of pH 4.0 to 4.5.

For this reason, sulfuric acid is required for pH control of the waste prior to add these coagulants.

In the stage of flocculation, however, dose of lime for controlling the pH value at 7 to 8 is better improving the floc-settling characteristics in the sedimentation tank.

Partial return of the settled sludge into the flush mixing tank is effective for the solid/liquid separation in the sedimentation tank.

v) It is desirable to set the surface loading of the sedimentation tank at $40m^3/m^2.d.$ or below, and the detention time at more than 2 hrs. or more.

vi) Liquid-state surface active agent separated from the foam separation tank is brought in contact with the settled sludge for a long period, and then almost entirely is removed as adsorbed into diatomaceous earth.

Fig. 4.5 shows a flow sheet of the joint pretreatment facility for the waste, which has been in operation since March this year.

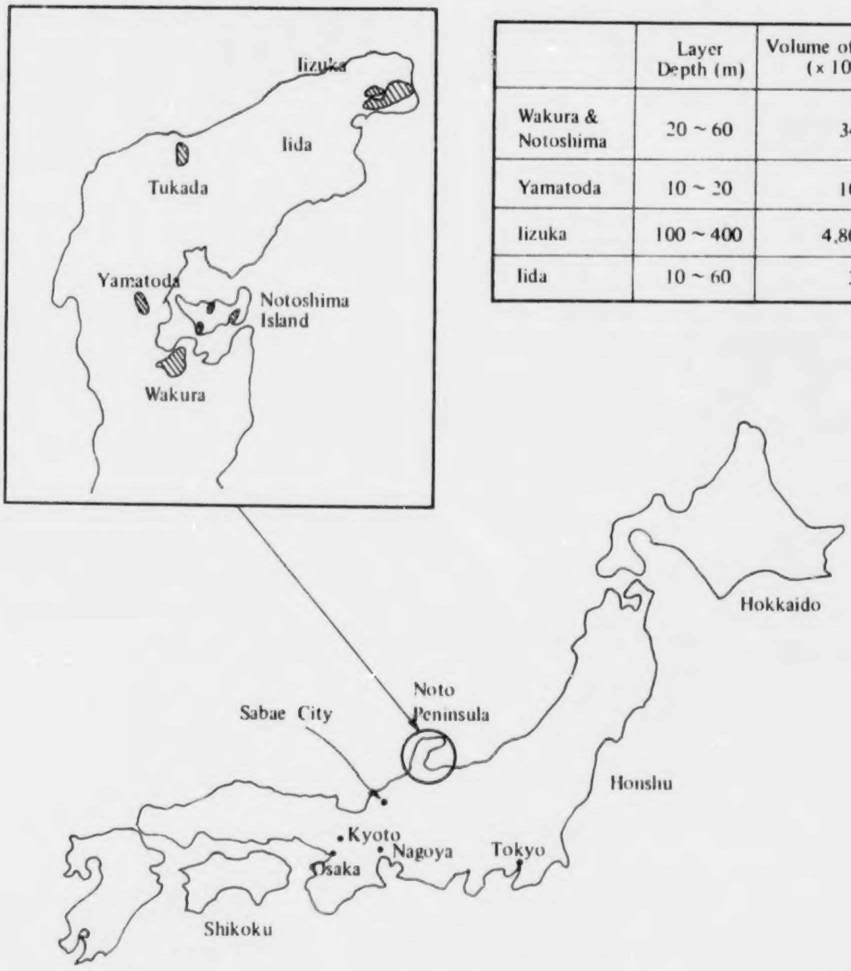
A part of the operation result are as shown in Table 4.2. Although the results of the treatment of actual wastes containing various kinds of dyestuffs are worse than those of the independent treatment of each dyestuff, but the results are still encouraging.

Table 4.1 COD Removals and Color Reduction Effects by Dosing Coagulant "Activated Diatom Earth" 0.5 g/L or 1.0 g/L

Items / Dyes	COD_{Mn} (PPM)	0.5 g/ℓ		1.0 g/ℓ	
		COD_{Mn} removal	Color reduction	COD_{Mn} removal	Color reduction
(Disperse dyes) (0.1 g/ℓ)		(%)	(%)	(%)	(%)
Resoline blue BR	80.8	67.3	100	84.9	100
Kayalon fast blue RD GGF	46.6	63.7	94.9	65.7	97.2
Kayalon polyester turg. blue	54.3	70.7	100	78.6	100
(Acid dyes) (0.1 g/ℓ)					
Lanasyn blue GL	77.4	91.1	100	92.5	100
Cidalan brill red RL	75.5	69.4	100	70.4	100
Suminol leveling red 3B	41.6	73.1	92.4	78.4	97.1
Kayanol milling red RS	24.7	88.3	100	95.6	100
(Naphtol dyes) (0.1 g/ℓ)					
TD black 10G-2S	65.1	84.8	100	86.6	100
(Direct dyes) (0.1 g/ℓ)					
Dialuminous blue GF	25.4	76.8	79.5	87.8	100
(Vat dyes) (0.1 g/ℓ)					
Indanthren red FBB	54.3	82.9	100	84.0	100
(Reaction dyes) (0.1 g/ℓ)					
Mikacion rubine BS	39.8	51.7	88.6	57.0	88.9
(Basic dyes) (0.1 g/ℓ)					
Aizen cathilon red GTLH	65.3	40.6	90.5	52.0	95.1

Table 4.2 A Part of Operational Data of Dying Waste Treatment Facility, Sabae City, Fukui Pref.

Items	Date Sampling time	Mar. 8 12:00	Mar. 8 14:00	Mar. 8 16:00	Mar. 10 13:30	Mar. 10 15:30	Mar. 13 13:00	Mar. 14 11:30	Mar. 14 13:30	Mar. 14 15:30
Temperature (°C)	Inf.	35.0	34.5	35.0	35.0	35.0	32.0	35.0	36.0	34.0
	Eff.	33.5	33.0	33.0	33.0	33.0	33.0	34.5	34.5	34.0
pH	Inf.	7.50	7.50	7.45	7.50	7.80	7.68	7.6	7.73	7.65
	Eff.	6.40	6.42	6.52	6.67	6.80	6.82	6.75	6.85	6.90
Transparency (cm)	Inf.	17	17	16	17	17	17	16	15	12.5
	Eff.	>30	>30	>30	>30	>30	>30	>30	>30	>30
$\mathrm{COD_{Mn}}$ (mg/ℓ)	Inf.	127	117	117	116	131	140	128	143	156
	Eff.	63.6	58.6	58.6	39.7	43.8	52.4	65.3	58.5	65.3
BOD (mg/ℓ)	Inf.	99	101	112	102	118	109	112	118	131
	Eff.	53	45	48	38	35	39	54	55	57
Sus. solids (mg/ℓ)	Inf.	13.3	15.2	16.2	20.4	21.6	20.0	26.3	24.7	20.3
	Eff.	12.4	7.2	6.2	5.2	3.2	6.4	23.4	17.2	17.2
400 mμ absorbance	Inf.	0.223	0.212	0.232	0.252	0.265	0.235	0.264	0.294	0.361
	Eff.	0.107	0.082	0.082	0.051	0.051	0.066	0.097	0.086	0.081


Iizuka
Iida
Tukada
Yamatoda
Notoshima Island
Wakura
Hokkaido
Noto Peninsula
Sabae City
Honshu
Kyoto
Nagoya
Tokyo
Osaka
Shikoku

	Layer Depth (m)	Volume of Deposits ($\times 10^6$ m^3)
Wakura & Notoshima	20 ~ 60	340
Yamatoda	10 ~ 20	100
Iizuka	100 ~ 400	4,800
Iida	10 ~ 60	25

Fig. 4.1 Destribution of Diatom Earth Raw Materials in Noto Peninsula, Ishikawa Prefecture

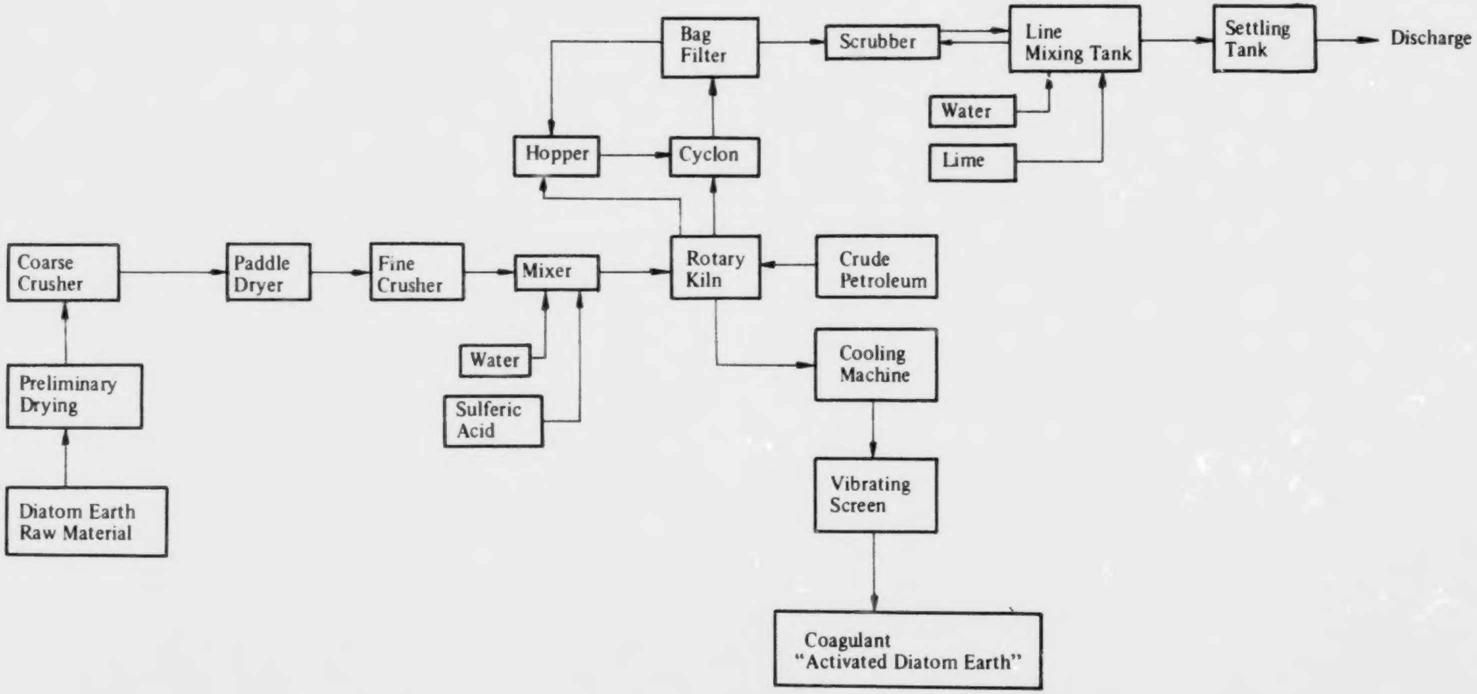

Bag Filter
Scrubber
Line Mixing Tank
Settling Tank
Discharge
Water
Lime
Hopper
Cyclon
Coarse Crusher
Paddle Dryer
Fine Crusher
Mixer
Rotary Kiln
Crude Petroleum
Water
Sulferic Acid
Cooling Machine
Preliminary Drying
Diatom Earth Raw Material
Vibrating Screen
Coagulant "Activated Diatom Earth"

Fig. 4.2 Manufacturing Flow Diagram of Coagulant "Activated Diatom Earth"

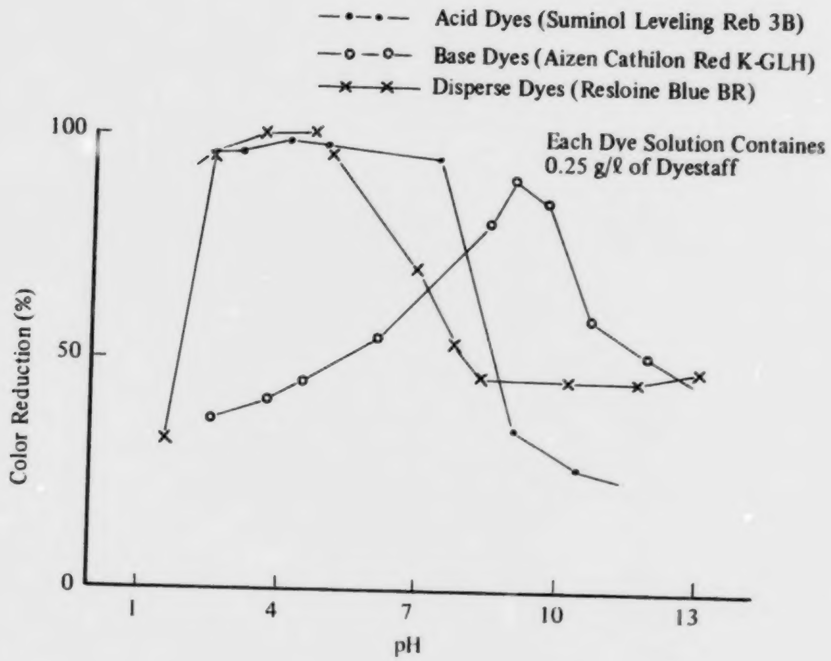


Fig. 4.3 Relationship between Color Reduction Effects and pH Variation

(Coagulant "Activated Diatom Ecarth" Dose 1 g/ℓ)

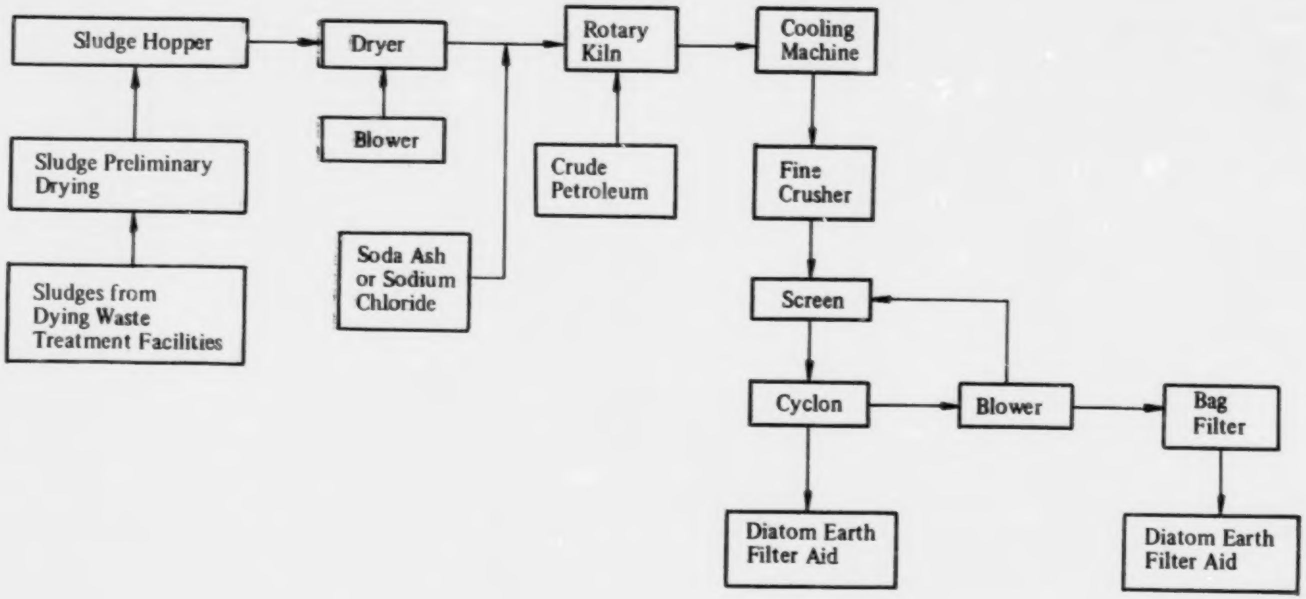

Sludge Hopper
Dryer
Rotary Kiln
Cooling Machine
Blower
Sludge Preliminary Drying
Crude Petroleum
Fine Crusher
Soda Ash or Sodium Chloride
Sludges from Dying Waste Treatment Facilities
Screen
Cyclon
Blower
Bag Filter
Diatom Earth Filter Aid
Diatom Earth Filter Aid

Fig. 4.4 Manufacturing Flow Diagram of Diatom Earth Filter Aid from Sludges from Dying Waste Treatment Facilities

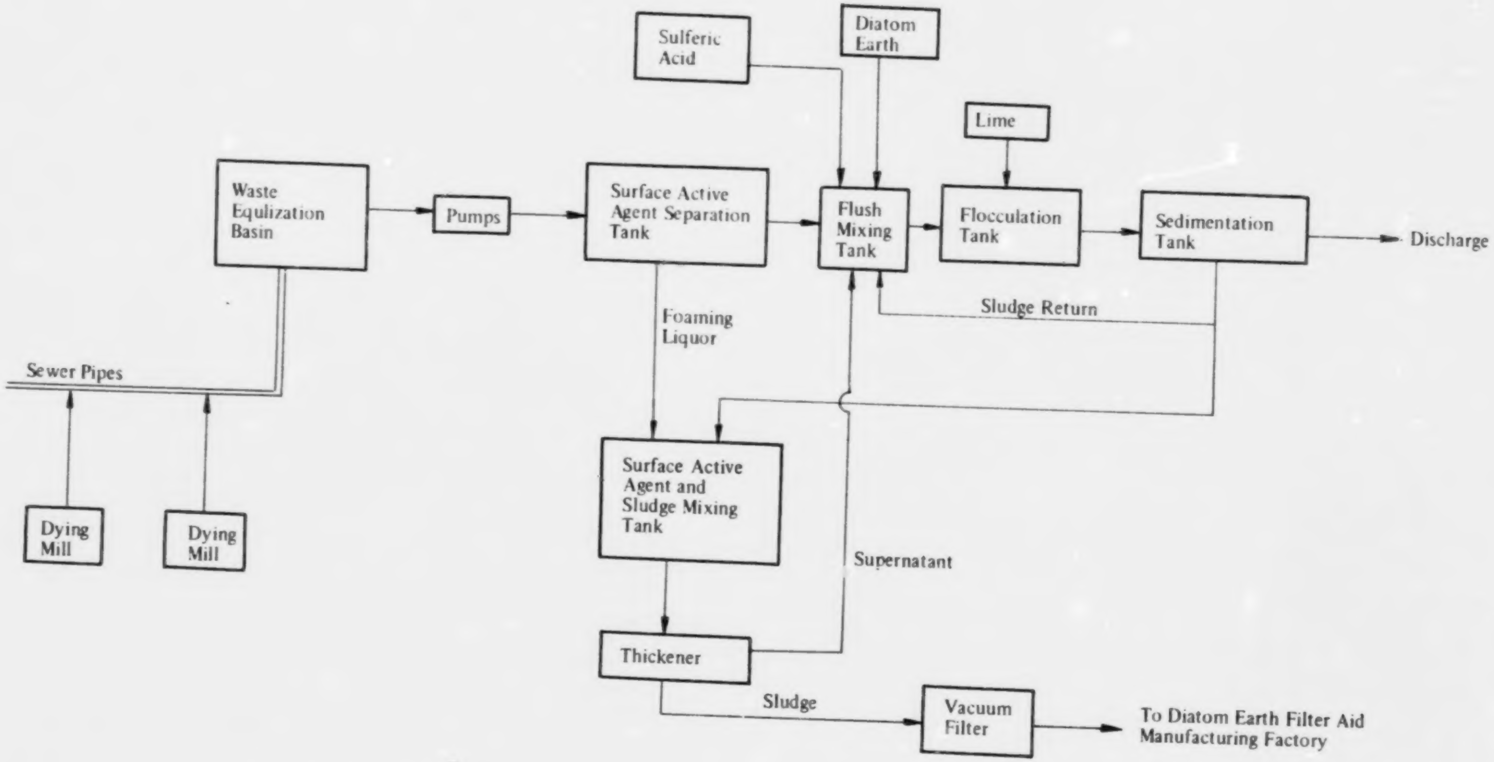

Sulferic Acid
Diatom Earth
Lime
Waste Equlization Basin
Pumps
Surface Active Agent Separation Tank
Flush Mixing Tank
Flocculation Tank
Sedimentation Tank
Discharge
Sludge Return
Foaming Liquor
Sewer Pipes
Surface Active Agent and Sludge Mixing Tank
Supernatant
Dying Mill
Dying Mill
Thickener
Sludge
Vacuum Filter
To Diatom Earth Filter Aid Manufacturing Factory

Fig. 4.5 Flow Diagram of Dying Mills Waste Treatment Plant, Sabae City, Fukui Prefecture

CHAPTER 5. IMPROVEMENT OF EFFLUENT QUALITY OF BISAI DISTRICT SEWAGE TREATMENT PLANT IN AICHI PREFECTURE

5.1 Quality of Sewage and Present State of the Sewage Treatment Plant 490
5.2 Installation of Pilot Plant 491
5.3 Installation of Pretreatment Facilities 491
5.4 Results of Laboratory and Pilot Plant Experiments 492
5.5 The Modification of Sewage Treatment Plant 494
5.6 Experiments and Studies of the Future 494

5. IMPROVEMENT OF EFFLUENT QUALITY OF BISAI DISTRICT SEWAGE TREATMENT PLANT IN AICHI PREFECTURE

5.1 QUALITY OF SEWAGE AND PRESENT STATE OF THE SEWAGE TREATMENT PLANT

Bisai City, Inchinomiya City and Kisogawa-cho, in Aichi Pref. are alive with one hundred and odd factories processing raw wool, dying and finishing woolen textiles, the wastes coming from which have been treated at the Bisai District Sewage Treatment Plant designed solely for industrial wastes treatment.

In the district, there are little factories which are equipped with a through production system handling all the way from raw wool processing to the finishing of woolen fabrics.

The production system is roughly classified into four processes: raw wool processing, dying, weaving and finishing.

Of them, weaving factories do not deliver industrial wastes, but all the others vomit wastes peculiar to respective processes.

The factories with raw wool process deliver wastes containing a great deal of organic substance and grease. The factories with dying process discharges wastes of residual dyes, inorganic salts (mainly sodium sulfate, soldium sulfide, etc.) and high temperature waste water containing various kinds of surface active agents. The factories with finishing process discharge waste water containing unrecoverable short fibers and various surface active agents.

The quality of raw sewage running into the Bisai Sewage Treatment Plant is shown in Table 5.1.

The raw sewage is almost neutral, but is high in temperature and contains reducing materials and greases.

The concentration of suspended solids is not so high, but most is occupied by short fibers which are hard to settle in the sedimentation tank.

The reducing materials are mainly composed of sulfides.

This is because in addition to sodium sulfide contained in sewage, sodium sulfate is reduced into sulfides biochemically in the sewer pipes. The Public Works Research Institute of the Ministry of Construction has once conducted an experiment on the reduction of sulfate ions of this waste water into sulfides.

The results are as shown in Fig. 5.1. The test temperature was 25 to 30°C.

The present facilities of the Bisai District Sewage Treatment Plant include three circular primary sedimentation tanks, three four-path rectangular aeration tanks, nine rectangular final settling tanks, one rectangular thickener, two vacuum filters, and a sludge incinerator.

This sewage treatment plant is located next to the Ichinomiya Seibu Sewage Treatment Plant, and some of the facilities are used common to both plants.

A plan view of the Bisai District Sewage Treatment Plant is shown in Fig. 5.2.

When designing this plant, the effects of short wool fibers, greases and oils, reducing materials and various surface active agents on the process were not evaluated thoroughly. In addition, sludge production rate and thickening rate were misevaluated.

Design capacity of the existing facilities is 70,000 m^3/d. On one occasion when the production of woolen products increased, the influent into the sewage plant reached as large as 140,000 m^3/d. At present, the influent is in the range of 80,000 to 100,000 m^3/d. as a result of strong save-water campaign against factories.

Even this, however, still surpasses the planned capacity of the plant.

While the effluent quality standard applied to this sewage treatment plant is 20 mg/l or less in BOD, effluent of the Plant is in the range of 60 to 80 mg/l.

At the request of the Sewage Management Authority, the Water Quality Control Division of the Public Works Research Institute, Ministry of Construction, is extending technical assistance in the improvement of effluent quality and modification of the plant facilities.

5.2 INSTALLATION OF PILOT PLANT

The request of the Sewage Management Authority for the improvement of effluent quality and modification of facilities presupposed increasing the design capacity up to 100,000 m^3/d. and reducing BOD of effluent to 20 mg/l. without expanding the existing site area.

In fact, the lots adjoining to the plant are hard to purchase, and the expansion of the right-of-ways for the plant from the present scale is still more difficult.

The open space available in the premises is very limited. For this reason, it was judged difficult to add up the primary sedimentation tanks, aeration tanks and final settling tanks.

In order to manage a proposed flow of 100,000 m^3/d. with the existing facilities, the design values should be reset as follows.

Average overflow rate of the primary sedimentation tank: 27.2 $m^3/m^2.d$.

Average aeration time: 3.85 hrs.

Detention time of sewage in the aeration tank with sludge return ratio at 30%: 2.95 hrs.

Average overflow rate of final settling tank (with sludge return ratio at 30%): 35.3 $m^3/m^2.d$.

Prior to mapping out modification plans for sewage treatment facilities, various investigations were required, but it was difficult to utilize part of the existing facilities for testing purposes.

Accordingly, it was decided to install a pilot plant copying the existing plant to a scale of 1/1,000.

The pliot plant, was designed to simulate the changes of that in the full-scale plant.

5.3 INSTALLATION OF PRETREATMENT FACILITIES

In view of the limited capacity of the sewage treatment plant and the need to improve the effluent quality without expanding its facilities, it was also needed to request the factories to moderate their waste discharge into the sewer.

The requests were as follows:

i) Each factory to make every effort to limit its daily discharge not to exceed the total factory discharge of 79,000 m^3/d. set by the Sewage Management Authority.

ii) Each raw wool processing factory to install a pretreatment facility to remove high concentration of n-hexane extracts contained in its effluent to 30 mg/l. or less.

iii) Each of those raw wool processing and dying factories which discharge waste water of 40°C or higher to install a heat exchanger to recover heat in order to decrease wastewater temperature before discharge into the sewer to lower than 40°C.

iv) Each chromic dye-using factory to install a pretreatment facility to remove chromium and limit the chromium content in effluent before discharge to lower than 0.5 mg/l.
If possible, such chromium removal facility should be used common to all chromic dye-using factories.

v) Each finishing factory to install a micro screen of 4 × 4 mm meshes in order to reduce the discharge of short wool fibers.

vi) Those factories which discharge waste water of more that 1,000 m^3/d to install a storage tank for the purpose of equalizing effluent.
At present, the equalization of effluent is undertaken by two storage tanks (7,000 m^3 and 5,000 m^3) and sewer pipes (8,000 m^3) (20,000 m^3 in total).
If the factory with more than 1,000 m^3/d of waste water is equipped with a storage tank the influent of the sewage treatment plant will be kept almost constant all the day.

5.4 RESULTS OF LABORATORY AND PILOT PLANT EXPERIMENTS

Both laboratory study and pilot plant experiments are still in progress. The findings obtained so far are as follows:

i) Chemical flocculation to remove reducing materials, oils and greases and short wool fibers in the primary sedimentation tank has been studied both on a laboratory scale and pilot plant scale.
It is found that the use of aluminum sulfate and lime in combination is most effective.
The required dose is 200 to 300 mg/l. of $Al_2(SO_4)_3$ and 100 to 120 mg/l. of $Ca(OH)_2$. In this process, the oils and greases in the waste water can be removed 70 to 80% in terms of n-hexane extracts. Also, the reducing materials can be removed more than 60% in terms of iodine demands. In addition, short wool fibers can be removed efficiently. The sludge volume developed from the process is 170 to 220 mg/l in terms of suspended solids, and the under flow concentration of the sedimentation tank is less than 0.5%
According to the cylinder test, the thickening of this sludge is very poor: even with a 24 hrs thickening, the reduction of the sludge volume is only

about 50%.

ii) As against the overflow rate of 27.2 m^3/m^2 .d. (max. 31.7 m^3/m^2 .d.), the plain sedimentation gave suspended solids removal of approx. 80 mg/l or some 47%.
The withdrawn sludge concentration was approx. 0.5%.
According to the sludge thickening test by cylinder, the sludge volume was reduced to some one-third in about 18 hrs of thickening time.

iii) Effluent of plain sedimentation was treated by activated sludge process with an average aeration time of 3.83 hrs.
In this case, the ratio of sludge return from the final settling tank was 25%.
The concentration of return sludge was very high enough to maintain MLSS in the aeration tank at level of 3,000 to 4,000 mg/l.
As a result, effluent BOD was 16 to 20 mg/l with MLSS above 3,000 mg/l, BOD loading lower than 0.376 kg/kg. MLSS.d. and with the air supply more than 9 times the influent.
In this case, SVI of the activated sludge was approx. 100. DO in the aeration tank was able to be kept at 2 mg/l all the time.
The results of molecular graduation of influent and effluent by Sephadex G15 are shown in Fige. 5.3 and 5.4.

iv) Sometimes, the influent carries wastes with a odor of naphtol. In such a case, DO in the aeration tank is reduced almost naught, and BOD of effluent becomes higher than 20 mg/l.
It remains uncertain what air flow rate can keep DO concentration in the aeration tank above 0.5 mg/l.
Molecular graducation by Sephadex G15 of influent and effluent in these cases is as shown in Figs. 5.5 and 5.6.

v) The influent shown in Fig. 5.5 is richer in low moleoular materials of fraction number 28 or above than that shown in Fig. 5.3.
High molecular weight materials are less compared with Fig. 5.3.
On the other hand, the influent shown in Fig. 5.3 is rich in high molecular weight materials of more than 1,500 in molecular weight and in colloidal materials.

vi) Fig. 5.4 indicates that organic removal was significant in the process.
The influent shown in Fig. 5.3 contained a great amount of high molecular weight meterials and colloidal materials, but did not affect to the treatment at all.
The peak of the ultea-violet absorbance curve for the effluent shown in Fig. 5.6 was shifted toward the lower molecualr weight side as compared with the effluent in Fig. 5.4. This is inferred to be ascribable to the fact that the dissolved oxygen might have been in deficit in the activated sludge process.

vii) Plastic diffuser were used for the pilot plant. The oxygen absorption efficiency of this plastic diffuser was 6.1% when the water depth was 2.0 m above the diffuser.

The disc diffuser now in operation creates some 3% of oxygen absorption efficiency when the water depth is 3 m. This low performance is further aggravated by the colgging of the pores with grease and short wool fibers contained in the sewage.

It is therefore urgently needed to replace the diffuser with one which is not affected by grease and short wool fibers and with which high oxygen absorption efficiency can be attained.

viii) The oxygen consumption rate, k_r. of the activated sludge was measured at four points in the aeration tank. The results were 37.0, 28.2, 20.9 and 15.9 mgO_2/gr/hr.

The selection of diffuser and determination of air supply rate are to be made according to the above results, and in consideration of the case where sewage of low oxygen absorption efficiency can run into the system on some occasions.

ix) There were lots of scum and floating matters on the surface of the final Settler.

The scum increased BOD and suspended solids of the effluent.

5.5 THE MODIFICATION OF SEWAGE TREATMENT PLANT

i) The primary sedimentation tank is to be modified to carry out plain sedimentation.

The sludge collector is to be modified to have an increased collecting speed in order to shorten the detention time of sludge in the tank.

The primary sedimentation tank is to be equipped with a scum collector. In order to improve the sludge withdrawal, the valves and pipeline are to be modified.

ii) The diffusers are to be replaced with a more efficient ones.

Additional air blowers are to be installed to make up air supply.

iii) The final settling tank is to be equipped with a scum collector. Also, an air lift is to be provided for easier withdrawal of sludge.

iv) The sludge thickener is to be expanded in order to allow more than 18 hrs of thickneing time.

It is also to be provided with a scum collector in the thickener.

5.6 EXPERIMENTS AND STUDIES OF THE FUTURE

i) Preaeration will be carried out for the purpose of increasing the grease removal by promoting dewatering of emulsified greases and oils and of increasing the removal of reducing materials.

The detention time of the preaeration tank should be within 20 min.

ii) Experiments for the determination of the diffuser characteristics to be applied in the aeration tank will be conducted. The experiments should cover determination of the oxygen transfer efficiency and the protection of the diffuser from clogging by grease.

iii) Study will be made to provide measures to maintain the level of dissolved oxygen constant in the aeration tank even when sewage of low oxygen demand

iv) Dewatering of thickened sludge will be studied on presupposition that the sludge be incinerated, and the equipment will be selected so to meet.

v) Odor control at preaeration tank and sludge treatment facilities will be studied.

Table 5.1 **Influent Quality of Bisai District Sewage Treatment Plant, Aichi Pref. (February to March 1975)**

Items \ Range or average		Range	Average
Sewage temperature	(°C)	21.0 ~ 31.5	27.5
Transparency	(cm)	2.2 ~ 5.7	3.6
pH		6.6 ~ 8.4	7.3
Suspended solids	(mg/ℓ)	50 ~202	129.0
COD_{Mn}	(mg/ℓ)	62.9 ~110.2	88.7
BOD	(mg/ℓ)	89.8 ~291.6	181.2
Iodine demands	(mg/ℓ)	8.3 ~ 46.4	28.2
n-Hexane extracts	(mg/ℓ)	7.6 ~128.0	70.3
Total chremium	(mg/ℓ)	0.21~ 1.10	0.59

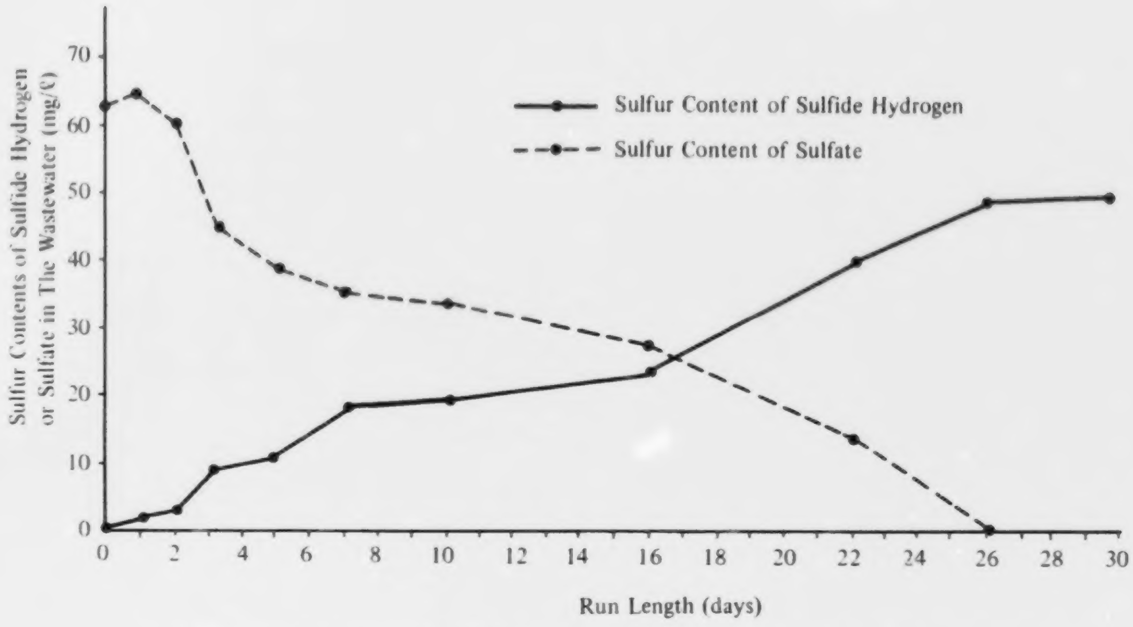


Fig. 5.1 Sulfur Contents Variation of Reducing and Oxdizing Compounds in the Wastewater (Laboratory Test)

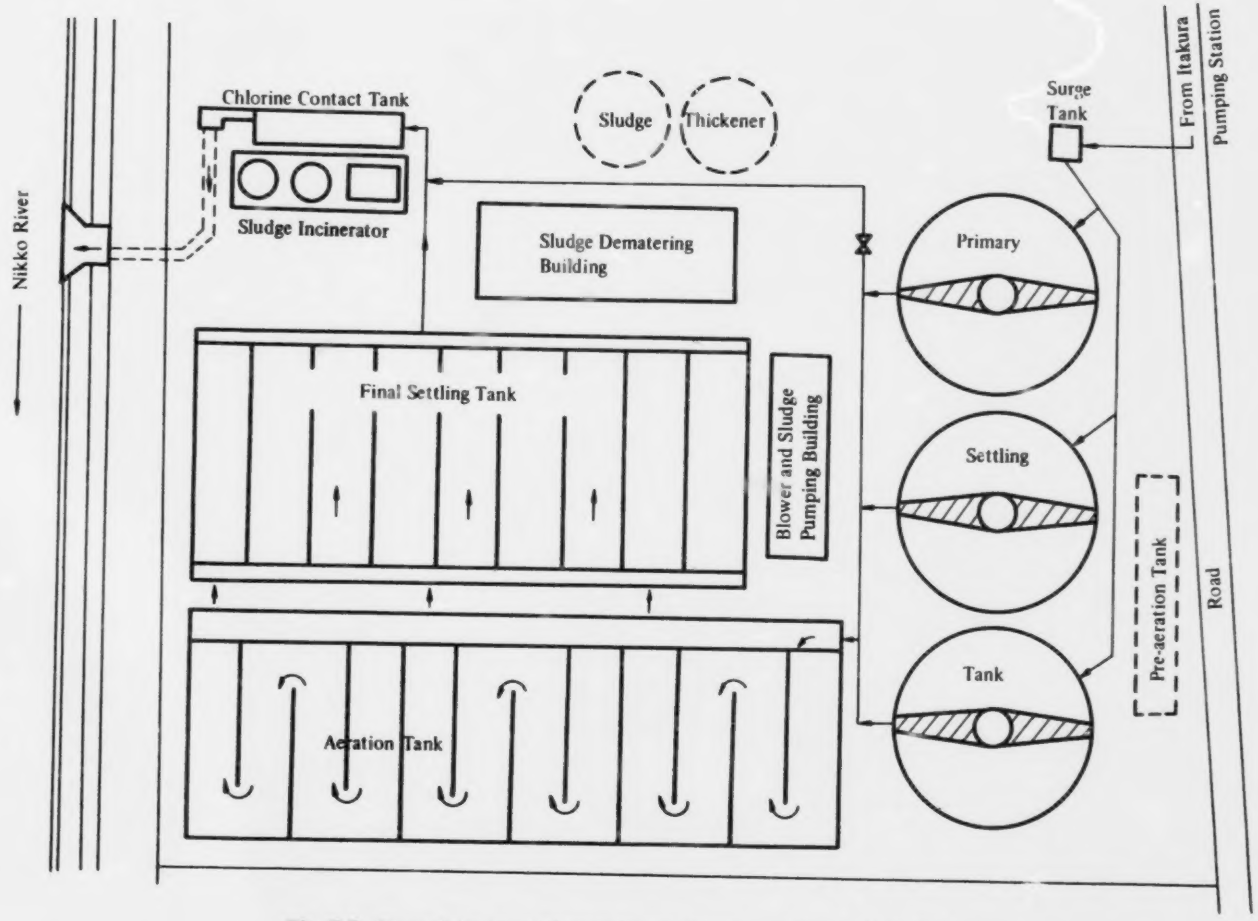

Chlorine Contact Tank
Sludge
Thickener
Surge Tank
From Itakura
Pumping Station
Nikko River
Sludge Incinerator
Sludge Dematering Building
Primary
Settling
Tank
Final Settling Tank
Blower and Sludge Pumping Building
Pre-aeration Tank
Road
Aeration Tank

Fig. 5.2 Plan of the Bisai District Sewage Treatment Plant, Aichi Prefecture

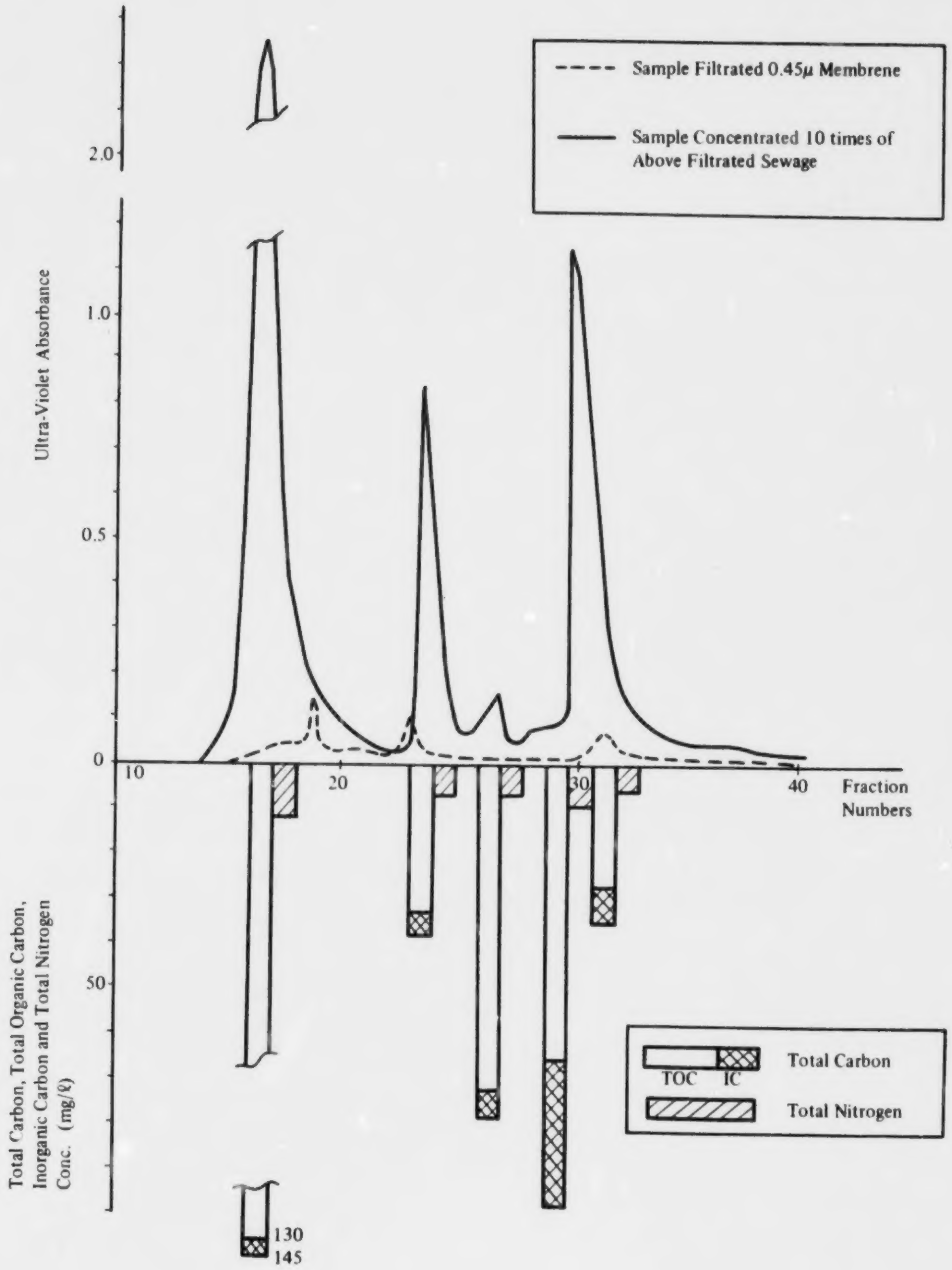


Fig. 5.3 Molecular Graduations of Influent, Ultra-Violet Absorbance, and Carbon and Nitrogen Compound Variations

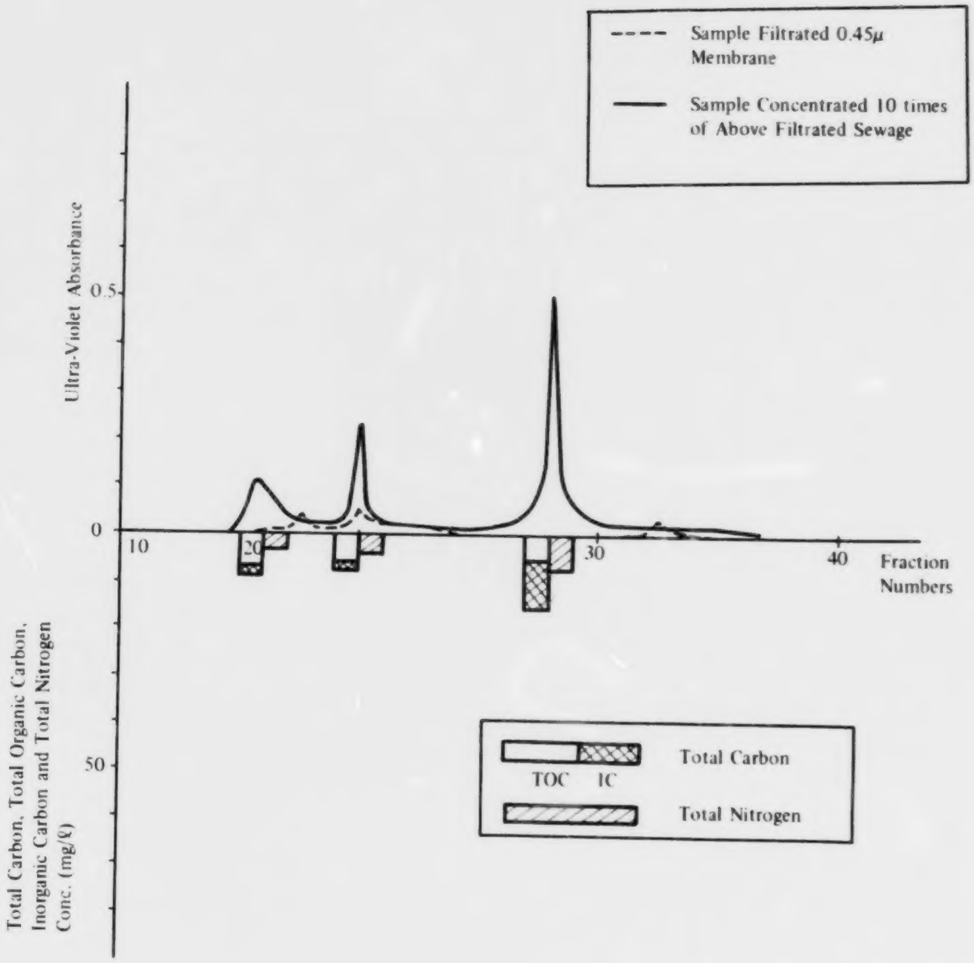


Fig. 5.4 Molecular Graduation of Effluent, Ultra-Violet Absorbance, and Carbon and Nitrogen Compound Variations

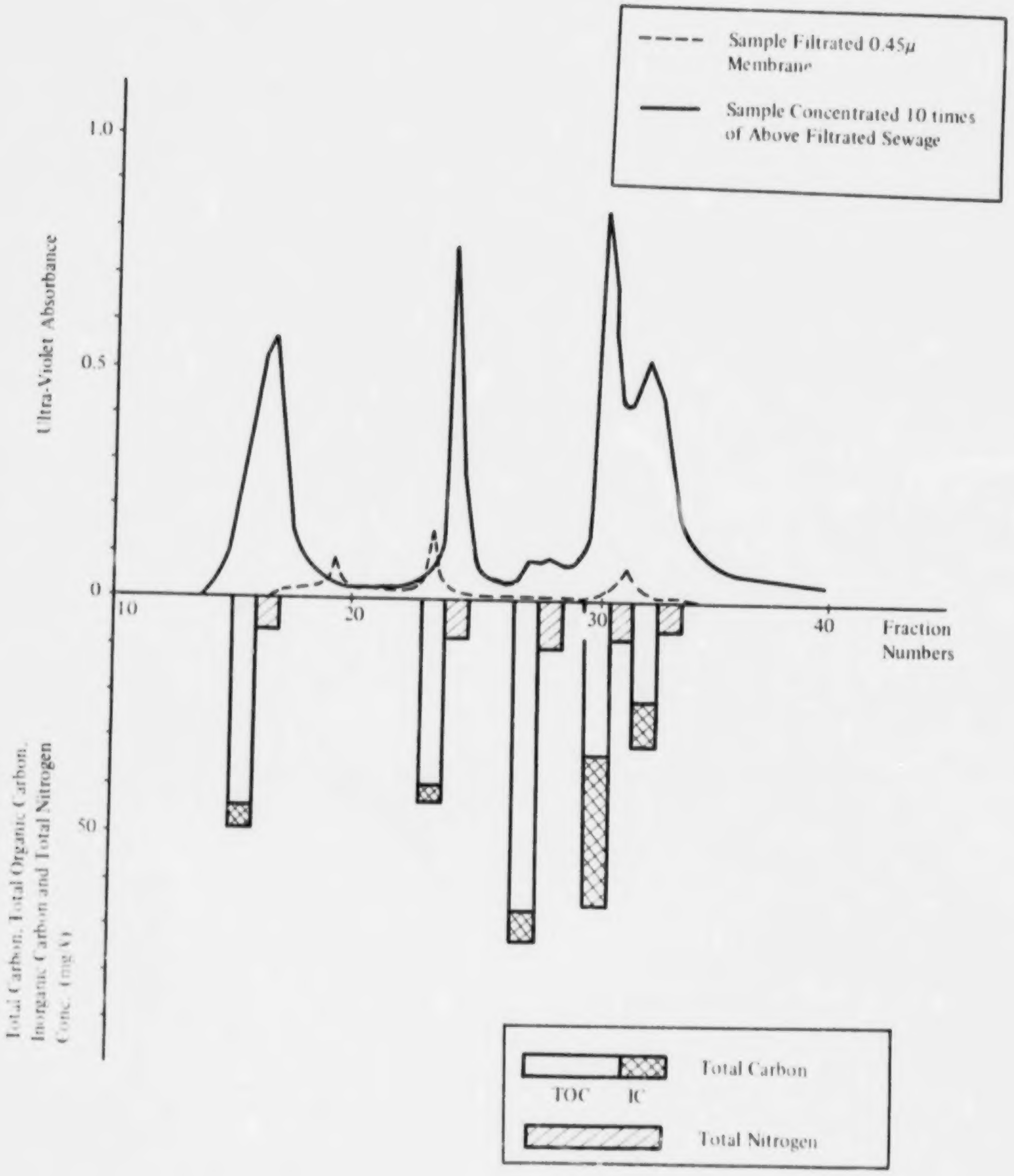


Fig. 5.5 **Molecular Graduation of Effluent, Ultra-Violet Absorbance, and Carbon and Nitrogen Compound Variations**

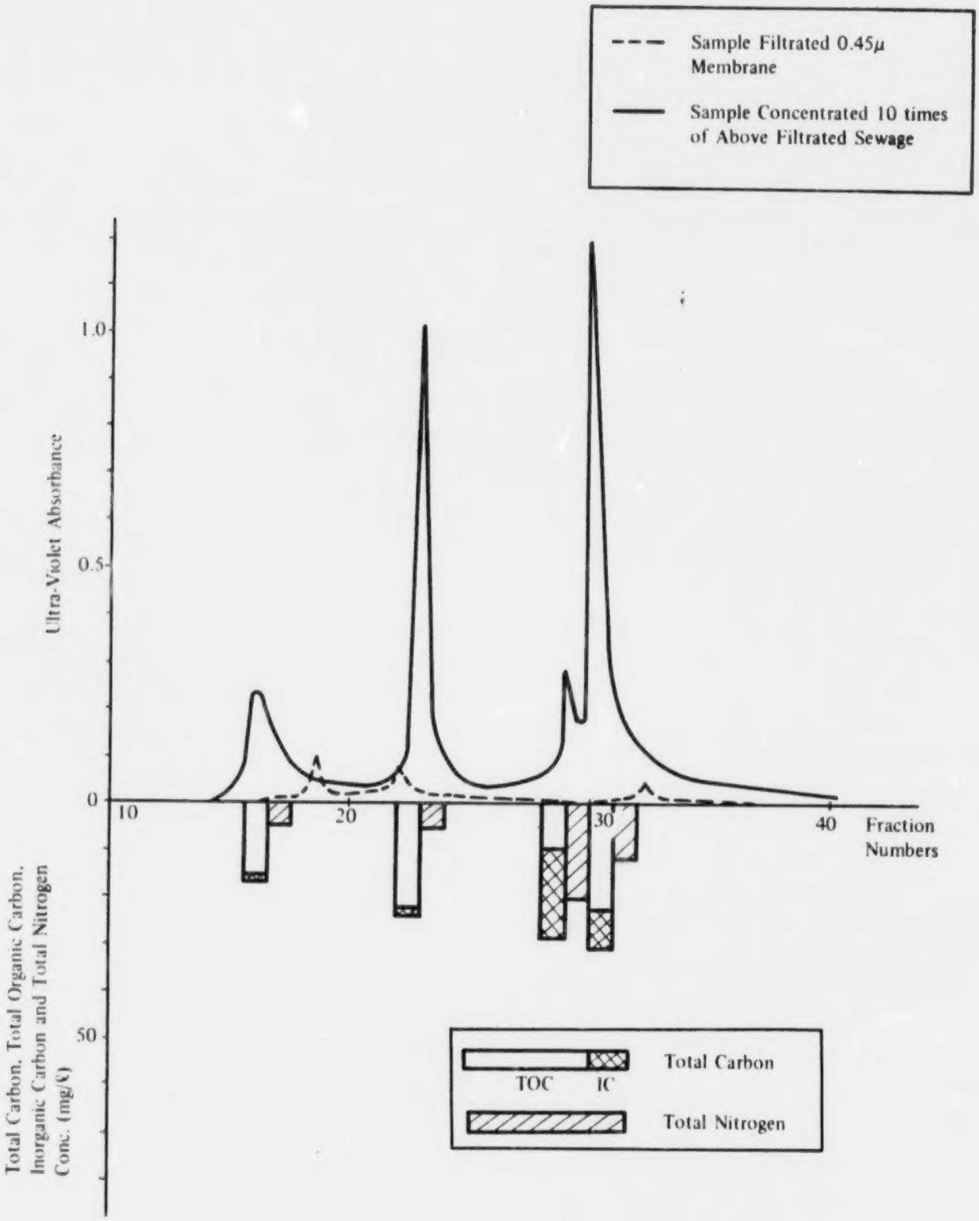


Fig. 5.6 Molecular Graduation of Effluent, Ultra-Violet Absorbance, and Carbon and Nitrogen Compound Variations

AGENDA

CINCINNATI, OHIO

ROBERT A. TAFT LABORATORY

Thursday, October 23

8:30 am	F. M. Middleton - Official Opening of Conference Dr. T. Kubo - Response
8:45 am	L. W. Lefke - Welcome to Cincinnati
8:55 am	F. M. Middleton - Agenda Approval
9:00 am	R. C. Brenner - Updated Status of Oxygen-Activated Sludge Wastewater Treatment
9:45 am	Break
10:00 am	B. V. Salotto - Current Research Related to Heat Conditioning of Wastewater Sludge
10:30 am	R. A. Olexsey - EPA's Research Program in Sewage Sludge Combustion
11:00 am	R. I. Field - Urban Stormwater Management and Technology in the United States - An Overview
12:00 noon	Lunch
1:30 pm	J. J. Westrick - Activated Carbon for Municipal Wastewater Treatment
2:15 pm	J. Ciancia - New Industrial Wastewater Separation Processes Developed Under the EPA Research Programs
3:00 pm	Break
3:15 pm	J. F. Roesler - Status of Instrumentation and Automation for Control of Wastewater Treatment Plants
4:00 pm	J. N. English - Research Required to Establish Confidence in the Potable Reuse of Wastewater
4:30 pm	J. J. Convery - Summary and Comments
5:00 pm	F. M. Middleton - Closing and Announcements

UPDATED STATUS OF OXYGEN-ACTIVATED SLUDGE WASTEWATER TREATMENT

R. C. Brenner
U. S. Environmental Protection Agency
Municipal Environmental Research Laboratory
Cincinnati, Ohio 45268

ABSTRACT

The oxygen-activated sludge wastewater treatment process continues to enjoy vigorous growth in the United States. Recently, new markets for oxygenation technology have emerged in Japan, Europe, and the greater North American continent. As of June 1976, 50 oxygenation installations were in operation. An additional 66 oxygen plants were under construction and designs were in progress for another 41. The combined design flow of these 157 known commitments to the use of oxygen exceeds 5.8 bgd (254 cu m/sec). Approximately one-half of the operating installations are industrial wastewater applications.

The Union Carbide Corporation, from its vantage point of first entry into the field, has collected the major share of the oxygen market to date with its covered reactor UNOX system. The only other available covered reactor oxygenation system is sold by Air Products and Chemicals, Inc. under the trade name OASES. An open reactor oxygen-activated sludge option (MAROX) employing ultra fine bubble rotating active diffusers has recently been developed by the FMC Corporation. A large-scale demonstration project evaluating this MAROX system is currently underway at Metropolitan Denver's main wastewater treatment plant. The project is a joint effort of the local government, FMC, and the federal government.

INTRODUCTION

In the past eight years, the use of oxygen gas in the activated sludge process has evolved from a level of primarily academic interest to a point of broad application and implementation. A large and rapidly growing number of oxygen-activated sludge plants are in operation in North America and Japan. Several plants will soon be operational in Europe. Included among the operating facilities are installations treating process wastewaters from six major industrial categories. By 1980, it is projected that construction will be completed on approximately 150 oxygen systems with a combined hydraulic capacity between 5 and 6 bgd (219 to 263 cu m/sec).

Beginning with the initial research project conducted by Union Carbide at Batavia, New York, in 1968 and 1969 (1), the development and refinement of oxygenation technology has been more rapid than normally associated with wastewater treatment processes. Design engineers today can select from several oxygen dissolution concepts including both covered and open reactor alternatives. The covered reactor UNOX and OASES systems are available with either surface aerators or submerged turbines. The surface aerator option has become the standard covered reactor design except in cases where unusually deep tanks are specified. FMC markets two versions of its open reactor MAROX system, one utilizing rotating active diffusers (RAD's), the other fixed active diffusers (FAD's). At this time, the second generation RAD design appears to be a significant cost-effective improvement compared to the original FAD design. In the near future, a fourth company, AIRCO Cryoplants, is expected to start actively competing in the oxygenation field. AIRCO has named its latest development the F^3O system, which stands for forced free fall oxygenation. The principal components of the F^3O system are a gas-tight drop-in concrete hood, an axial flow pump, a nozzle, and an oxygen enriched waterfall zone.

The purposes of this paper are three-fold:

1. To provide an updated status report on the number and type of oxygen-activated sludge facilities in operation, under construction, and being designed.

2. To describe in detail the latest EPA supported oxygenation research and demonstration project, an evaluation of the RAD version of the open reactor MAROX system being carried out at the Metropolitan Denver, Colorado Sewage Treatment Plant.

3. To summarize design, operating, and performance information for several on-line oxygen wastewater treatment systems.

STATUS REPORT - JUNE 1976

A complete listing of the 50 oxygen-activated plants that were in operation as of June 1976 is presented in Appendix A. The 66 oxygenation plants under construction on the same date are listed in Appendix B. An additional 41 oxygenation plants were in various stages of design during June 1976; these plants are listed in Appendix C. Besides these 157 known commitments to oxygen use, proposals have been made to numerous other potential municipal and industrial customers by the several oxygen proprietary firms. These potential customers are not included in Appendix C because final decisions on process selection have not yet been made. It should be understood from the above comments that the status of oxygen implementation is in a state of flux and that the three lists given in the Appendices will be out of date within several months.

All three appendices provide design flow and oxygen supply data (where known) for each plant location listed, as well as identifying the wastewater application. Multiple oxygen process applications, such as carbonaceous organics removal plus nitrification, aerobic digestion, ozonation, etc., are also noted where applicable. In addition, Appendices A and B include information on the oxygen dissolution and oxygen supply systems selected. This latter information is not given in Appendix C because these plants have either not yet been bid or litigation has delayed awarding of contracts to specific oxygen system suppliers.

Perusal of Appendix C reveals that no industrial wastewater applications are shown in the "plants being designed" list. This omission is not intended to indicate that no industrial plants were in the design phase as of June 1976, but rather that the identity of such plants is confidential proprietary information until after equipment purchase contracts are awarded.

Data on the number of plants, design flows, and oxygen supply capacities have been extracted from Appendices A, B, and C and condensed in Table 1. The same information is presented in Table 2 for United States oxygen plants only. These two tables indicate that as of June 1976 only about 12 percent of the firm planned oxygen design flow capacity was actually completed and in operation. On-line capacity is expected to increase 7-8 times, however, in the next 4-5 years. Approximately 25 percent of the oxygen installations included in the Table 1 totals are treating or will treat industrial process wastewaters. Excluding the Japanese plants for which oxygen supply data were unavailable to the writer, the design oxygen supply capacity averages 3.07 tons/mil gal of design flow (7.4×10^{-4} metric ton/cu m) for the industrial applications compared to 1.34 tons/mil gal of design flow (3.2×10^{-4} metric ton/cu m) for the municipal applications.

A breakdown of the 157 known operating and planned oxygen installations by country is given in Table 3. Eighty-five percent of these installations are or will be located in the United States and 11 percent in Japan. The remaining 4 percent are divided among seven other countries with one plant each.

The detailed information provided in Appendices A and B on oxygen dissolution and oxygen generation systems is summarized in Tables 4 and 5, respectively. The dominance of Union Carbide in the oxygen dissolution market to date is evident: of the 116 oxygen installations that were operational or under construction in June 1976, a UNOX system was selected for 87 percent. In most but not all cases, the vendor supplying the oxygen dissolution equipment was also awarded the oxygen

TABLE 1. WORLDWIDE OXYGEN PLANT STATUS - JUNE 1976

Parameter	Operating Plants	Plants Under Construction	Plants Being Designed	Total
No. of Plants				
Municipal	26	49	41	116
Industrial	24	17	--	41
Total	50	66	41	157
Design Flow (mgd)*				
Municipal	584.5	2302	2647.3	5533.8
Industrial	135.5	191.5	--	327.0
Total	720.0	2493.5	2647.3	5860.8
O_2 Supply Capacity† (tons/day)‡				
Municipal	477.7	3126.5	3794	7398.2
Industrial	389	462.5	--	851.5
Total	866.7	3589.0	3794	8249.7

*1 mgd = 0.044 cu m/sec

†Oxygen supply figures shown do not include data for Japanese plants; these data were unavailable to the writer.

‡1 ton/day = 0.907 metric ton/day

TABLE 2. USA OXYGEN PLANT STATUS - JUNE 1976

Parameter	Operating Plants	Plants Under Construction	Plants Being Designed	Total
No. of Plants				
Municipal	21	46	41	108
Industrial	14	11	--	25
Total	35	57	41	133
Design Flow (mgd)*				
Municipal	570.3	2172.8	2647.3	5390.4
Industrial	94.3	166.8	--	261.1
Total	664.6	2339.6	2647.3	5651.5
O_2 Supply Capacity (tons/day)†				
Municipal	467.7	2944.5	3794	7206.2
Industrial	389	383.8	--	772.8
Total	856.7	3328.3	3794	7979.0

*1 mgd = 0.044 cu m/sec

†1 ton/day = 0.907 metric ton/day

TABLE 3. BREAKDOWN OF OXYGEN PLANTS BY COUNTRY - JUNE 1976

Country	No. of Plants			
	Operating	Under Construction	Being Designed	Total
1. USA	35	57	41	133
2. Japan	14	3		17
3. Canada	1			1
4. Mexico		1		1
5. England		1		1
6. Germany		1		1
7. Denmark		1		1
8. Switzerland		1		1
9. Belgium		1		1
Total	50	66	41	157

supply system contract. The preponderance of surface aerators over submerged turbines in covered reactor systems is illustrated in Table 4 and is attributed to the lower overall costs and maintenance requirements of the aerator option. Surface aerators are being or will be used in 93 percent of the covered reactor systems with specified dissolution equipment, submerged turbines in 6 percent, and a combination of both in one system. Plants employing submerged turbines have deep aeration tanks, typically greater than 20 ft (6.1 m), and tend to be larger than 100 mgd (4.4 cu m/sec) in size. Conversely, the average design flow of the 103 surface aerator systems is only about 18 mgd (0.8 cu m/sec).

Cryogenic oxygen gas generators are sold by several firms in the United States, whereas Union Carbide is the only known USA manufacturer of pressure swing adsorption (PSA) oxygen gas generators. The break-even range determined by Union Carbide for these two oxygen supply systems is approximately 20-25 tons/day (18-23 metric tons/day). Below this range, it is more cost effective to use PSA generators; above this range, cryogenic generators are more cost effective. Other manufacturers have developed mini-cryogenic oxygen generators to compete for the lower tonnage plants. On-site cryogenic or PSA gas generation was selected for 80 percent of the 99 oxygen-activated sludge plants with defined methods of oxygen supply, as of June 1976. The average capacities for these 79 supply systems are 92 tons/day (83 metric tons/day) for the cryogenic units and 16.2 tons/day (14.8 metric tons/day) for the PSA units.

Pipeline transport of off-site generated oxygen gas to an oxygenation wastewater treatment plant can be an economical choice of oxygen supply if the logistics are reasonable and if the off-site facility (e.g., a steel production plant) has extra generation capacity. This method of oxygen supply accounts for 9 percent of the defined supply systems and 6 percent of the June 1976 "operating" and "under construction" capacity. On-site storage and vaporization of trucked-in liquid oxygen, because of its high unit cost, is generally confined to requirements of 5 tons/day (4.5 metric tons/day), or less. The oxygen consumption of the 11 such systems documented in Table 5 is expected to average 2.9 tons/day (2.6 metric tons/day); this amounts to only 0.7 percent of the defined oxygen supply capacity.

TABLE 4. SUMMARY OF OXYGEN DISSOLUTION SYSTEM VENDORS, TYPES, AND DESIGN FLOWS - JUNE 1976

Parameter	Operating Plants	Plants Under Construction	Total
O_2 Dissolution System Vendor (No.)			
UNOX - Covered Reactor	44	57	101
OASES - Covered Reactor	4	6	10
MAROX - Open Reactor	2	3	5
Total	50	66	116
O_2 Dissolution System Type (No.)			
Covered - Surface Aerators	44	59	103
Covered - Submerged Turbines	4	3	7
Covered - Combination of Aerators and Turbines	0	1†	1
Open - Rotating Active Diffusers	2	2	4
Open - Fixed Active Diffusers	0	1	1
Total	50	66	116
O_2 Dissolution System Design Flow (mgd)*			
Covered - Surface Aerators	286	1526	1812
Covered - Submerged Turbines	423.8	345	768.8
Covered - Combination of Aerators and Turbines	0	600†	600
Open - Rotating Active Diffusers	10.2	21.5	31.7
Open - Fixed Active Diffusers	0	1	1
Total	720.0	2493.5	3213.5

*1 mgd = 0.044 cu m/sec

†The oxygen dissolution system for Detroit's second-phase construction consists of submerged turbines in the lead stages and surface aerators in the rear stages.

TABLE 5. SUMMARY OF OXYGEN SUPPLY SYSTEM TYPES AND CAPACITIES - JUNE 1976

Parameter	Operating Plants	Plants Under Construction	Total
O_2 Supply System Type (No.)			
On-Site Cryogenic Generation	7	31	38
On-Site PSA Generation	15	26	41
On-Site Liquid Storage and Vaporization	9	2	11
Off-Site Pipeline Transport	5	4	9
Unknown*	14	3	17
Total	50	66	116
O_2 Supply System Capacity (tons/day)†			
On-Site Cryogenic Generation	407	3086.8	3493.8
On-Site PSA Generation	254	413.5	667.5
On-Site Liquid Storage and Vaporization	27.8	4.5	32.3
Off-Site Pipeline Transport	178	84.2	262.2
Total	866.8	3589.0	4455.8

*Data unavailable for Japanese oxygen supply systems

†1 ton/day = 0.907 metric ton/day

Oxygen plants treating or scheduled to treat industrial wastewaters are broken down by industrial application in Table 6. Eight major categories are represented in the "operating" and "under construction" classifications. The pulp and paper industry leads the list: nearly one-half of the total plants and over three-fourths of the total design flow. The next most frequent users to date have been the petrochemical and chemicals industries. Inasmuch as oxygenation technology is well suited to satisfying the high oxygen demand associated with many industrial wastewaters, continuing rapid growth in the oxygen industrial market is anticipated for years to come.

DESCRIPTION OF SECOND GENERATION FMC OPEN REACTOR OXYGEN SYSTEM (MAROX)

The covered reactor oxygen system, including both the surface aerator and submerged turbine alternatives, has been described previously in the Proceedings of the Second U.S.-Japan Conference on Sewage Treatment Technology (2) and elsewhere (3). A description of the first generation fixed active diffuser (FAD) version of the open reactor oxygenation system was also provided in these documents. It is not deemed necessary to reiterate those descriptions here; however, certain characteristics of the covered reactor systems and the FAD open

TABLE 6. BREAKDOWN OF OXYGEN PLANTS BY INDUSTRIAL APPLICATION

Industrial Application	Operating Plants No. of Plants	Operating Plants Design Flow (mgd)*	Plants Under Construction No. of Plants	Plants Under Construction Design Flow (mgd)*	Total No. of Plants	Total Design Flow (mgd)*
1. Chemicals	4	10.9	3	15.2	7	26.1
2. Dyestuffs	0	0	1	3.1	1	3.1
3. Food Processing	1	1	1	1.8	2	2.8
4. Petrochemical	5	9.9	3	15.0	8	24.9
5. Pharmaceutical	2	1.7	0	0	2	1.7
6. Pulp & Paper	11	111.2	8	142.7	19	253.9
7. Steel	0	0	1	13.7	1	13.7
8. Synthetic Rubber	1	0.8	0	0	1	0.8
Total	24	135.5	17	191.5	41	327.2

*1 mgd = 0.044 cu m/sec

reactor system are compared with the second generation open reactor FMC option, described below.

A section view of the key element (the rotating active diffuser (RAD)) of the second generation FMC system is shown in Figure 1. As indicated, the basic RAD consists of a 7-ft (2.1-m) diameter submerged rotating plate mounted to the bottom of a 6 5/8-inch (16.8-cm) diameter hollow shaft approximately 3 ft (0.9 m) above the aeration tank floor. A 7 1/2-inch (19.1-cm) wide ceramic diffusion medium is inserted into preformed openings top and bottom around the periphery of the plate, forming two circular diffusion bands parallel to the outer tapered edge. Approximate 28-inch (71-cm) diameter radial impellers mounted to the top and bottom of the plate provide essential mixing of oxygen, substrate, and biomass. An optional surface impeller can be installed to aid in foam breakup, if desired. The relatively low design rotational velocity of 75-85 rpm is achieved with a constant speed motor and an appropriate gear reduction unit. The composite submerged assembly is illustrated in a cutaway perspective view in Figure 2.

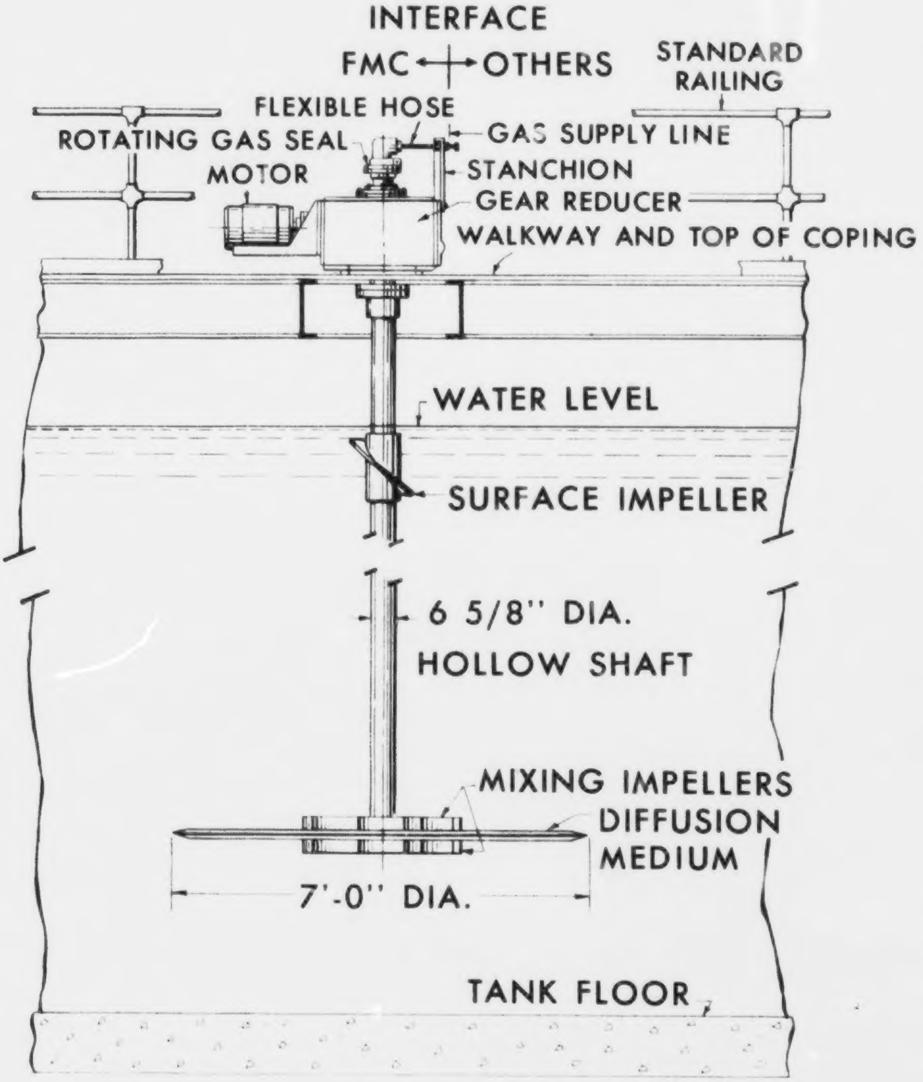

INTERFACE
FMC ← → OTHERS
STANDARD RAILING
FLEXIBLE HOSE
ROTATING GAS SEAL
GAS SUPPLY LINE
MOTOR
STANCHION
GEAR REDUCER
WALKWAY AND TOP OF COPING
WATER LEVEL
SURFACE IMPELLER
6 5/8" DIA.
HOLLOW SHAFT
MIXING IMPELLERS
DIFFUSION MEDIUM
7'-0" DIA.
TANK FLOOR

Figure 1. Section view of rotating active diffuser and drive assembly. (Printed, with modifications, through the courtesy of the FMC Corporation.)

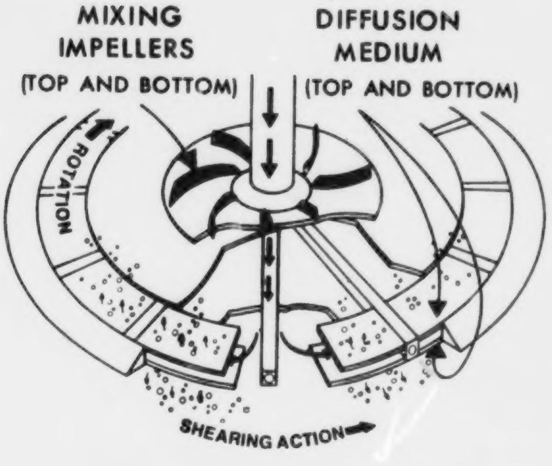

MIXING
IMPELLERS
(TOP AND BOTTOM)
DIFFUSION
MEDIUM
(TOP AND BOTTOM)
ROTATION
SHEARING ACTION

Figure 2. Perspective view of submerged rotating active diffuser showing gas flow and bubble formation. (Reprinted, with modifications, through the courtesy of the FMC Corporation.) (4)

A functional flow diagram for a typical MAROX system employing RAD's for oxygen transfer is presented in Figure 3. The primary oxygen supply (shown as a cryogenic generator) is supplemented by a liquid oxygen reserve supply and accompanying vaporizer. With a cryogenic generator, unlike a PSA generator, losses occurring from the liquid oxygen backup tank, either through usage or evaporation, can be replenished directly from the primary supply source.

Oxygen gas from the supply system is pressurized to 30 psig (2.1 kgf/sq cm) with a separate compressor (not shown in Figure 3) and fed down through the hollow RAD shafts and then radially outward through small ducts located inside the diffuser plate to the ceramic medium. As oxygen gas emerges from the upper and lower diffusion bands, the rotational shear created by centrifugal force forms ultra small bubbles in the 50-100 micron range which do not coalesce as they move outward and pass over the outside tapered edge of the diffuser plate. The primary function of the tapered edge is to prevent turbulence which could induce bubble coalescence. The resulting micron bubble dispersion resembles a mist from which oxygen is rapidly and efficiently dissolved in the mixed liquor. The oxygen transfer rate obtained with bubbles of this minute size is sufficiently high to reportedly sustain an oxygen utilization efficiency greater than 90 percent in conventional depth uncovered aeration tanks (4).

A dissolved oxygen (DO) feedback system is used to control the oxygen feed rate to the RAD's. The control system, consist-

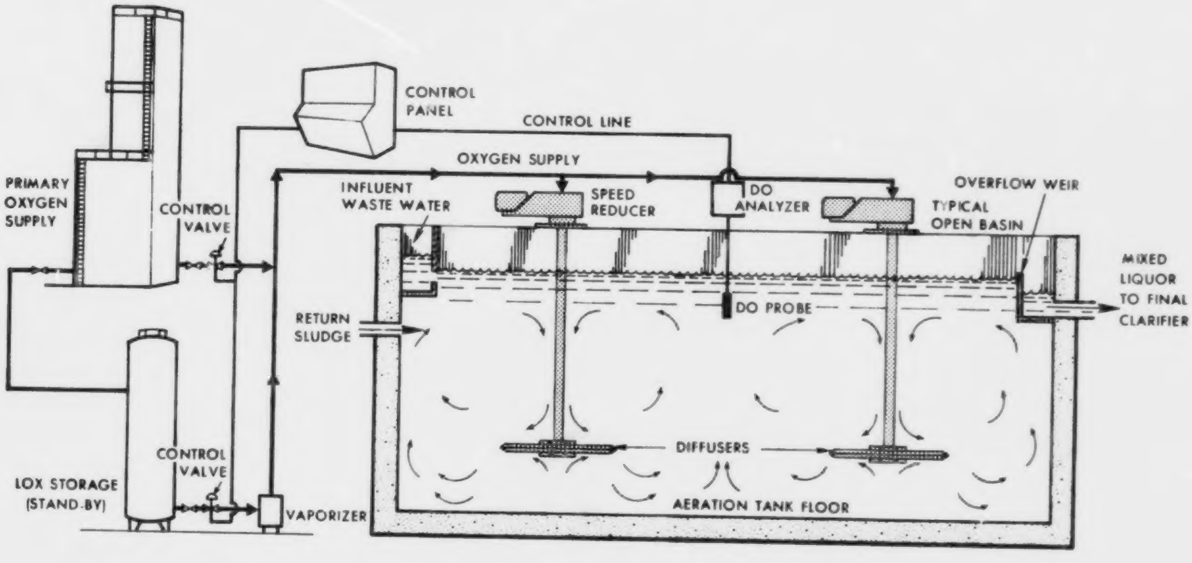

CONTROL
PANEL
CONTROL LINE
OXYGEN SUPPLY
PRIMARY
OXYGEN
SUPPLY
CONTROL
VALVE
INFLUENT
WASTE WATER
SPEED
REDUCER
DO
ANALYZER
OVERFLOW WEIR
TYPICAL
OPEN BASIN
MIXED
LIQUOR
TO FINAL
CLARIFIER
RETURN
SLUDGE
DO PROBE
DIFFUSERS
CONTROL
VALVE
LOX STORAGE
(STAND-BY)
VAPORIZER
AERATION TANK FLOOR

Figure 3. Functions flow diagram of typical MAROX system employing rotating active diffusers. (Reprinted, with modifications, through the courtesy of the FMC Corporation.) (4)

ing of one or more DO probes, analyzers, control valves, and electronic controllers, automatically maintains the mixed liquor DO concentration at a predetermined setpoint, within the tolerance range of the equipment. A one-module control system, i.e., one probe, analyzer, control valve, and controller each, is shown controlling the oxygen feed rate to both diffusers in Figure 3. In a longer tank requiring 10-20 RAD's, multiple control modules would be necessary with each module controlling the feed rate to a bank of 3-5 diffusers.

The lack of necessity for a tank cover enables FMC to avoid the sealing problems with its MAROX systems that must be considered with the covered reactor systems. MAROX systems do not utilize internal staging baffles. Although most covered reactor systems designed to date have included staging baffles, they are not essential. Both the open and covered reactor alternatives can be designed compatibly with any of the commonly used activated sludge flow regimes. Covered reactor systems are, however, more naturally adapted to the conventional plug flow regime. Where conventional activated sludge treatment is the flow regime of choice, the staged configuration more nearly approximates ideal plug flow and, other factors being equal, would be expected to deliver an effluent with a slightly lower soluble BOD than an unstaged system.

Other features distinguishing the open and covered reactor approaches from each other are:

1. The type of oxygen feed control systems. As mentioned previously, MAROX utilizes a DO based oxygen feed control system. The covered reactor systems control oxygen feed rate by maintaining a predetermined gas pressure in the first-stage head space.

2. Freeboard requirements. Covered reactors require more freeboard than open reactors. The greater freeboard is needed to provide adequate gas space for the umbrella throw pattern of the surface aerators normally employed in covered reactor designs. Utilization of submerged oxygen dissolution equipment obviates the necessity for as large a freeboard with the MAROX approach.

3. Carbon dioxide buildup. It is anticipated that MAROX systems will be less subject to carbon dioxide buildup and attendant pH depression than UNOX and OASES systems due to the absence of a tank cover. The degree to which cell respiration byproducts are vented from the open MAROX reactor will depend primarily on surface turbulence levels and the thickness of foam buildup, if any, on the aerator surface.

4. Hydrocarbon buildup. The absence of a tank cover virtually eliminates the possibility of accumulating an explosive concentration of volatile hydrocarbons over the aerator liquid surface. It is assumed, therefore, that safety precautionary measures could be less extensive with MAROX systems than with covered reactor systems.

5. Oxygen feed pressure to the oxygen dissolution systems. The nominal pressure of oxygen gas leaving cryogenic and PSA generators is 3-5 psig (0.21-0.35 kgf/sq cm). This is more than sufficient to satisfy line and entrance losses to a UNOX or OASES reactor and maintain a pressure of 1-3 inches (2.5-7.5 cm) of water in the first-stage vapor space. Conversely, head loss through either of the MAROX diffusers is substantial, requiring an additional compressor to pressurize generator output to 30 psig (2.1 kgf/sq cm).

In comparing FMC's two open reactor options, the several inherent advantages of the RAD system over the FAD system are expected to produce a pronounced preference for the RAD alternative. These advantages include:

- no requirement for prescreening of aerator influent,
- no requirement for pumping mixed liquor through the diffusers to create the necessary shear to produce micron size bubbles,
- reduced oxygen dissolution power requirements,
- simplified installation, and
- less maintenance.

METRO DENVER DEMONSTRATION PROJECT

In June 1975, the U.S. Environmental Protection Agency (EPA) awarded a $200,000 demonstration grant to Metropolitan Denver (Colorado) Sewage Disposal District No. 1 to evaluate the MAROX system. The remainder of the estimated total project cost of $605,000 is being shared by the District and FMC. The EPA Grant No. is S803910.

The evaluation is being conducted in a segment of Metro Denver's existing air-activated sludge plant. The plant's secondary system consists of thirty-six 210-ft (64-m) long, 670,000-gal (2536-cu m) aeration bays and twelve 130-ft (39.6-m) diameter clarifiers. Each of the clarifiers is mated with three aeration bays operated in series to form 12 parallel secondary trains. Several of the bays have on occasion been utilized for aerobic stabilization of waste activated sludge. Sludge is recycled separately for each quadrant of the plant, i.e., settled sludge from the three clarifiers in any given quadrant is transferred to a common collection well from where it is returned for distribution among the three aeration trains in that quadrant.

Approximately two-thirds of the average influent flow of 140 mgd (6.1 cu m/sec) receives primary sedimentation before it reaches the plant; the other third is primary settled on site. A new 72-mgd (3.2 cu m/sec) UNOX facility, scheduled to become operational in the fall of 1976, will divert a significant fraction of the primary effluent flow from the existing overloaded air-activated sludge plant.

Prior to grant award, it was mutually decided that the large-scale MAROX system to be evaluated by the District would employ RAD's rather than the older FAD's used in previous pilot-scale studies at Metro Denver and on a previous EPA supported grant project at the Englewood, Colorado, wastewater treatment plant (2)(3). Thirteen RAD's were installed in the first bay of aeration train No. 11 of the existing Metro air plant. The other two bays of this train have been taken out of service for the duration of the project. Required hydraulic modifications included the installation of a pipe to transfer mixed liquor from the end of the first bay to clarifier No. 11 and separate return and waste sludge lines and pumps. The latter step was taken to isolate MAROX sludge from the recycle sludge of the two remaining operating air trains (Nos. 7 and 9) of the plant's northeast quadrant. A liquid oxygen storage tank and vaporizer were installed adjacent to the converted oxygen test bay. During the first portion of the evaluation, trucked-in liquid oxygen is being used for oxygen supply. However, the two 40-ton/day (36.3-metric ton/day) cryogenic oxygen gas generators that will serve the new Metro Denver UNOX treatment plant will have excess capacity initially. For economic reasons, consideration is being given to utilizing the excess capacity for supplying oxygen to the MAROX demonstration project once shakedown of the cryogenic units is complete. If this action is taken, a compressor will have to be installed to raise generator output pressure to a level compatible with RAD operation. A process schematic of the Metro Denver MAROX test system is given in Figure 4. Dimensioned plan and section views are shown in Figure 5.

As indicated in Figure 5, the RAD's are located on 21-ft (6.4-m) centers. Six of the 13 diffusers were installed in sets of two in the first quarter of the tank where oxygen demand is greatest. The remaining seven diffusers are located in tandem on the longitudinal center line of the aeration tank. The first 11 RAD's are driven by 10-hp (7.5-kw) motors and rotate after gear reduction at 85 rpm. The motors for the last two RAD's are 7 1/2 hp-(5.6-kw) units. The rotational speeds of the twelfth and thirteenth RAD's are 80 and 76 rpm, respectively. The oxygen dissolution capability of the diffusers is rated at 1500 lb/day (680 kg/day) each in the District's wastewater for a total system capacity of 9.75 tons/day (8.85 metric tons/day). Previous proprietary tests indicated these diffusers can be operated up to 33 percent over their rated capacity without significantly affecting oxygen transfer efficiency. On this basis, assuming an average BOD_5 removal of 140 mg/l and an oxygen requirement of 1.3 lb O_2/lb BOD_5 removed (1.3 kg/kg), the maximum sustained flow which can be handled by this oxygen dissolution equipment is roughly 17 mgd (0.74 cu m/sec).

Three DO probes and control systems are employed to control oxygen feed to the Metro Denver test bay. One system controls the feed rate to the first six diffusers, the second to the middle four diffusers, and the

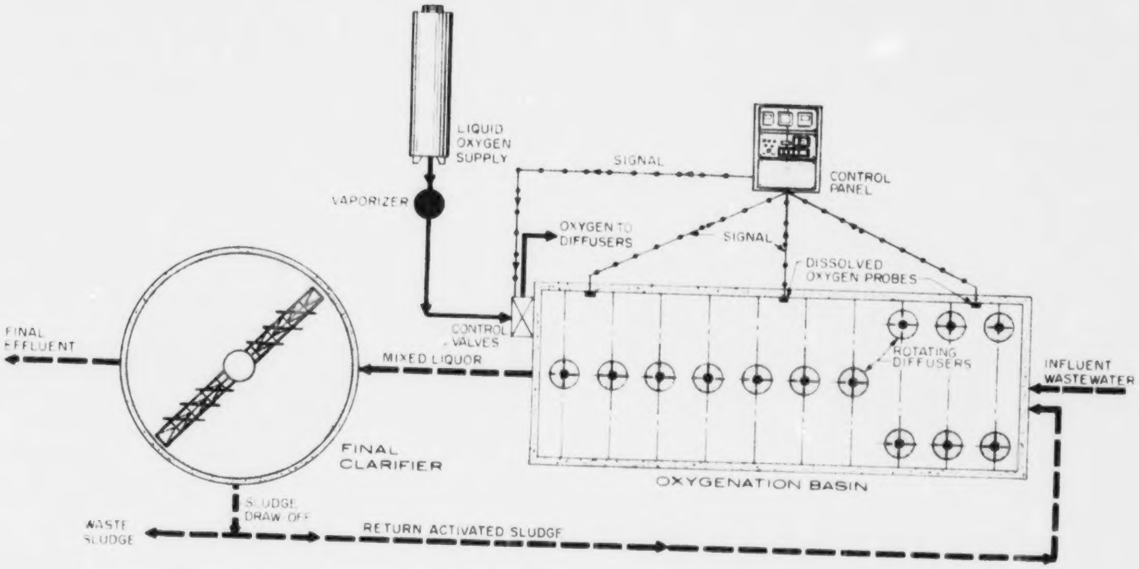

LIQUID OXYGEN SUPPLY
VAPORIZER
SIGNAL
CONTROL PANEL
OXYGEN TO DIFFUSERS
SIGNAL
DISSOLVED OXYGEN PROBES
CONTROL VALVES
FINAL EFFLUENT
MIXED LIQUOR
ROTATING DIFFUSERS
INFLUENT WASTEWATER
FINAL CLARIFIER
OXYGENATION BASIN
SLUDGE DRAW-OFF
WASTE SLUDGE
RETURN ACTIVATED SLUDGE

Figure 4. Process schematic of Metro Denver MAROX test system. (Reprinted through the courtesy of the FMC Corporation.) (5)

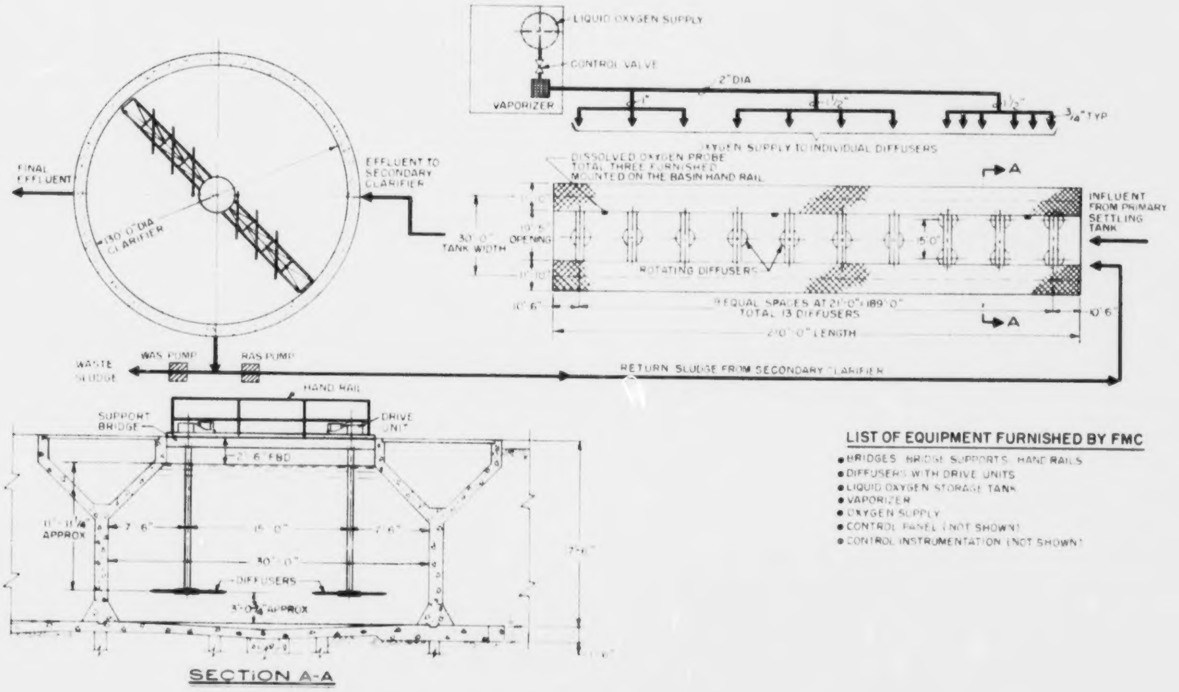

LIQUID OXYGEN SUPPLY
CONTROL VALVE
VAPORIZER
2" DIA
1"
1 1/2"
1 1/2"
3/4" TYP
OXYGEN SUPPLY TO INDIVIDUAL DIFFUSERS
DISSOLVED OXYGEN PROBE TOTAL THREE FURNISHED MOUNTED ON THE BASIN HAND RAIL
A
FINAL EFFLUENT
EFFLUENT TO SECONDARY CLARIFIER
130'-0" DIA CLARIFIER
30'-0" TANK WIDTH
10'-5" OPENING
15'-0"
ROTATING DIFFUSERS
INFLUENT FROM PRIMARY SETTLING TANK
10'-6"
9 EQUAL SPACES AT 21'-0" (189'-0") TOTAL 13 DIFFUSERS
10'-6"
210'-0" LENGTH
WASTE SLUDGE
WAS PUMP
RAS PUMP
RETURN SLUDGE FROM SECONDARY CLARIFIER
HAND RAIL
SUPPORT BRIDGE
DRIVE UNIT
2'-6" FBD
11'-11 1/4" APPROX
7'-6"
15'-0"
7'-6"
30'-0"
DIFFUSERS
3'-0 3/4" APPROX
SECTION A-A
LIST OF EQUIPMENT FURNISHED BY FMC
● BRIDGES, BRIDGE SUPPORTS, HAND RAILS
● DIFFUSERS WITH DRIVE UNITS
● LIQUID OXYGEN STORAGE TANK
● VAPORIZER
● OXYGEN SUPPLY
● CONTROL PANEL (NOT SHOWN)
● CONTROL INSTRUMENTATION (NOT SHOWN)

Figure 5. Dimensioned plan and section views of Metro Denver MAROX test system. (Reprinted through the courtesy of the FMC Corporation.) (5)

third to the last three diffusers. Based on mutual agreement, an initial DO setpoint of 3.0 mg/l was selected. During the first month following startup, the oxygen control equipment exhibited a variance range of ±0.7 mg/l from the desired setpoint.

The RAD's and RAD drives are supported from metal bridges which span the aeration test bay, as illustrated in Figure 5. The bridges in turn are supported by stanchions (not shown in Figure 5) running to the tank floor. The bridges were tied with minimal defacing into the side walls of the test bay to prevent lateral movement. Following delivery of the key components of the oxygen supply and dissolution systems to the project site, the entire installation including piping modifications was completed in six weeks. Due in part to the short period in which its system components can be installed and the minimum structural modifications required, the upgrading of existing air-activated sludge plants as exemplified by the Metro Denver demonstration project is expected to become an important MAROX application.

From the section view of Figure 5, it can be seen that surface impellers were not provided with the RAD's. The District has experienced a float buildup of relatively high solids concentration (2-3 percent TSS) on the mixed liquor surface. Under other circumstances and with the proper removal equipment, this float would constitute a potentially attractive source from which to waste excess sludge at a substantially higher solids concentration than available in secondary clarifier underflow. Since the District is not equipped to waste sludge in this manner, the presence of the float represents an operational and esthetic liability. To overcome this problem, installation of an aeration test bay overflow weir, similar to the one shown in Figure 3, is under consideration. The weir would replace the present submerged orifice through which the mixed liquor now exits the aeration bay. Utilization of an overflow weir would promote continuous transfer of floated solids to the secondary clarifier before they could accumulate on the liquid surface. Another float avoidance technique being evaluated is the use of one or more down draft propeller pumps to recirculate floated solids back into the mixed liquor. For long term operation, the overflow weir option is believed to be a more positive and cost-effective method than either surface impellers on the RAD shifts or down draft propeller pumps. For expediency on this finite length demonstration project, however, the down draft propeller pump technique may be selected, even though it would add 6-12 percent to oxygen dissolution system power requirements. A decision will be made in time for implementation during the month of September 1976.

The major objective of the project from the District's standpoint is to determine the technical feasibility and attendant costs of converting its existing air-activated sludge plant to a higher capacity (i.e., two to three times higher) open reactor, oxygen-activated sludge system. If successful, the District could potentially avert another major secondary plant expansion for the foreseeable future, with the exception of the additional clarifiers which would be needed to handle increases in influent flow. EPA's primary project objectives are: (1) to demonstrate at a representative field scale an alternative oxygenation concept which has been extensively and successfully evaluated at pilot scale and (2) to define reliable design criteria, operating conditions and costs, and performance expectation for a system embodying that concept for use by the engineering community.

Equipment installation and piping modifications were completed in early May 1976. The remainder of the month was devoted to facility shakedown and adjustments. June was utilized as a process start-up period for training operators and refining a data logging and retrival system. The evaluation program was initiated on July 1, 1976, and will continue for 10 months till April 30, 1977. The five planned phases and corresponding dates of the evaluation program are described below:

Phase I, July 1976,
    Constant flow @ 2 mgd (0.39 cu m/sec); warm wastewater temperatures; one clarifier only in use
Phase II, August-September 1976,
    Diurnally varied flow @ 7 to 14 mgd (0.31 to 0.61 cu m/sec); warm wastewater temperatures; second clarifier available, if necessary
Phase III, October 1976,
    Constant flow @ 2 mgd (0.34 cu m/sec); cool wastewater temperatures; one clarifier only in use

Phase IV, November-December 1976,
Diurnally varied flow @ 7 to 14 mgd (0.31 to 0.61 cu m/sec); cool wastewater temperatures; second clarifier available, if necessary
Phase V, January-April 1977,
Constant flow increased in increments to failure; cool wastewater temperatures; two clarifiers in use

Anticipated operating conditions are not documented here for each planned phase because of the variability that will be introduced by diurnal flow. However, for reference purposes, baseline operating conditions are summarized below for the 9-mgd (0.4 cu m/sec) constant flow phases, assuming an average primary effluent BOD_5 concentration of 140 mg/l, a sludge return rate equal to 40 percent of the influent flow rate, and average mixed liquor suspended solids (MLSS) and mixed liquor volatile suspended solids (MLVSS) concentrations of 4000 and 3200 mg/l, respectively:

Nominal Aeration Time (based on Q)
= 1.79 hr
Actual Aeration Time (based on Q + R)
= 1.28 hr
Food to Microorganism (F/M) Loading
= 0.59 lb BOD_5 applied/day/lb MLVSS under aeration (0.59 kg/day/kg)
Volumetric Organic Loading
= 117 lb BOD_5 applied/day/1000 cu ft aerator volume (1503 kg/day/cu m)
Secondary Clarifier Overflow Rate (based on total surface area)
= 678 gpd/sq ft (27.6 cu m/day/sq m)
Secondary Clarifier Overflow Rate (based on useful surface area; excludes effluent launder area)
= 746 gpd/sq ft (30.4 cu m/day/sq m)
Secondary Clarifier Mass Loading (based on floor area)
= 31.7 lb MLSS/day/sq ft (155 kg/day/sq m)

Average operating and performance data for the startup month of June 1976 are presented in Table 7. The average secondary effluent suspended solids (TSS) concentration of 30 mg/l is only marginally acceptable. Daily log sheets reveal, however, that this effluent parameter exhibited a steadily decreasing concentration trend throughout the 30-day period as operators became more familiar with system operation and sludge inventory management. Effluent TSS for the first 12 days of July averaged 20 mg/l, a 33 percent decrease from June. The seven-day/week data collection program depicted in Table 7 will be used, along with several additional tests not conducted in the startup month, throughout the planned evaluation studies. One of these additional tests will be the periodic determination of oxygen utilization efficiency. This will be accomplished with the aid of a 6-ft x 6-ft (1.8-m x 1.8-m) floating dome. Off gases from a 36-sq ft (3.34-sq m) area of tank surface will be collected inside the dome and funneled through a gas flow and composition monitoring station. The tent will be moved to different sections of the aeration test bay to arrive at a composite or average utilization efficiency.

Caution should be exercised in extrapolating the sludge production and oxygen supply rates given in Table 7. These values are for one month of operation only and were generated immediately following a period of operator familiarization with a new process. A better perspective of the relationship of these important parameters to organic loading will be gained from an evaluation of all the data at the end of the project.

CASE HISTORIES

Operating and performance data and case history summaries are presented below for 11 oxygen-activated sludge plants. Ten of the plants utilize UNOX oxygenation systems, the other an OASES system. All 11 plants are documented in the listing of operating facilities provided in Appendix A.

The case histories were selected to illustrate a variety of process applications, system component configurations, and plant sizes. Eight of the selected plants treat municipal wastewaters; three are strictly industrial applications. Several of the municipal installations receive a significant fraction of their incoming loads from industrial sources. The reactor designs for these plants represent a variety of configurations including both rectangular-stage systems and systems incorporating circular and arcuate stages within larger self-contained circular tanks.

In addition to operating and performance data, a flow diagram is presented for each case history along with pertinent background information, where known, leading to the selection of an oxygen system.

TABLE 7. JUNE 1976 AVERAGE OPERATING AND PERFORMANCE DATA FOR METRO DENVER OPEN REACTOR OXYGENATION PROJECT

Influent Flow	9.5 mgd (0.42 cu m/sec)
Return Sludge Flow	3.8 mgd (0.17 cu m/sec)
Return Sludge Flow/Influent Flow	40%
Pri. Eff. BOD_5	126 mg/l
Sec. Eff. BOD_5	19 mg/l
BOD_5 Removed Across Secondary	85%
Pri. Eff. TOC	87 mg/l
Sec. Eff. TOC	29 mg/l
TOC Removed Across Secondary	67%
Pri. Eff. TSS	88 mg/l
Sec. Eff. TSS	30 mg/l
TSS Removed Across Secondary	66%
MLSS	3050 mg/l
MLVSS	2610 mg/l (volatile fraction = 85.6%)
Mixed Liquor DO	2.7 mg/l
Mixed Liquor Temperature	20°C
Return Sludge TSS	10,970 mg/l
Return Sludge VSS	9120 mg/l (volatile fraction = 83.1%)
Depth to Clarifier Sludge Blanket	7.5 ft (2.3 m)
Nominal Aeration Time (based on Q)	1.69 hr
Actual Aeration Time (based on Q + R)	1.21 hr
F/M Loading	0.68 lb BOD_5 applied/day/lb MLVSS under aeration (0.68 kg/day kg)
Volumetric Organic Loading	111 lb BOD_5 applied/day/1000 cu ft aerator volume (1432 kg/day/cu m)
Secondary Clarifier Overflow Rate (based on total surface area)	716 gpd/sq ft (29.2 cu m/day/sq m)
Secondary Clarifier Overflow Rate (based on useful surface area; excludes effluent launder area)	787 gpd/sq ft (32 cu m/day/sq m)
Secondary Clarifier Mass Loading (based on floor area)	25.5 lb MLSS/day/sq ft (124 kg/day/sq m)
Waste Activated Sludge Mass	5060 lb/day (2295 kg/day)
Sludge Production Rate (based on waste sludge TSS only)	0.60 lb TSS/lb BOD_5 removed (0.60 kg/kg)
Sludge Production Rate (based on waste sludge & sec. eff. TSS)	0.88 lb TSS/lb BOD_5 removed (0.88 kg/kg)
Sludge Retention Time (SRT)	2.3 lb MLSS under aeration/(lb waste sludge TSS + sec. eff. TSS lost)/day = 2.3 days
RAD Power Draw	109 hp (81 kw)
Oxygen Supplied	11,363 lb O_2/day (5154 kg/day)
Oxygen Supply Rate (based on load)	1.14 lb O_2/lb BOD_5 applied (1.14 kg/kg)
Oxygen Supply Rate (based on removal)	1.34 lb O_2/lb BOD_5 removed (1.34 kg/kg)

Noteworthy start-up, operating, and maintenance difficulties encountered are discussed. Secondary system components and any flow routing peculiarities are described briefly. Data available to the writer for summarization herein varied from one month's results at several plants to more than two years' results at another location.

Decatur, Illinois (UNOX)

Prior to the recent addition of a UNOX system, the Sanitary District of Decatur's wastewater treatment plant consisted of two rectangular primary clarifiers, six Imhoff tanks, 12 air aeration bays, three secondary clarifiers, two trickling filters, one primary anaerobic digester, one secondary digester, one supernatant holding tank, and tertiary and sludge lagoons. Six of the existing air aeration bays are of 1935 vintage; the other six are larger and were installed in 1965.

In July 1975, the liquid portion of a comprehensive plant upgrading program was completed. The heart of this upgrading effort was the conversion of three of the 1965 air aeration bays to oxygen service. The walls of these bays were extended upwards 4 ft (1.2 m) and the bays covered to provide the needed vapor space to satisfactorily control oxygen feed and interstage gas transport. The remaining nine air aeration bays have been combined into an integrated system to operate in parallel with the oxygen unit in either the conventional mode or as a modified contact stabilization process. The nine bays are shown schematically in the flow diagram of Figure 6 as two tanks, one representing the six older 1935 bays, the other the three newer 1965 bays.

Coinciding with the modifications to implement oxygen-activated sludge treatment, three new primary clarifiers and four new secondary clarifiers were constructed. Two of the three new primaries have 100-ft (30.5-m) diameters and are in use continuously. The third new primary has a diameter of 130 ft (39.6 m) and is only used during severe storms with the overflow discharged directly to the receiving river following chlorination. The new secondary clarifiers are mated with the UNOX system, the old secondaries with the revamped air aeration facilities. The diameter and side water depth (SWD) of the new secondaries are 100 ft (30.5 m) and 12.5 ft (3.8 m), respectively. The old trickling filters (not shown in Figure 6) were abandoned in September 1975. The old rectangular primary clarifiers (also not shown in Figure 6) have been placed on standby service.

A program to upgrade the sludge handling portion of the plant is currently underway and is scheduled for completion in December 1976. The old supernatant holding tank and old secondary digester are being converted to heated primary anaerobic digesters to join the one existing primary digester. When completed, supernatant will be returned directly to the plant headworks. Five of the existing six Imhoff tanks (omitted from Figure 6) are being outfitted with covers to operate as non-heated secondary digesters. The sixth Imhoff tank will remain uncovered and serve as a holding tank for both oxygen and air waste activated sludges prior to separate thickening in a new concentrator. Waste sludge is presently returned to the primaries for thickening before digestion.

Each of the three oxygen trains is divided into four stages. The overall dimensions of the oxygen reactor are 148 ft long x 77 ft wide x 14 ft SWD (45 m x 23.5 m x 4.3 m) with a freeboard of 4 ft (1.2 m). The oxygen dissolution system consists of surface aerators combined with bottom propellers for additional mixing. The PSA oxygen generation unit has a design output capacity of 17 tons/day (15.4 metric tons/day). The storage capacity of the backup liquid oxygen supply tank is 43 tons (39 metric tons).

On the average, 55 to 60 percent of the incoming organic load is from industrial sources, primarily corn and soybean processing. Some of the industrial contributors have their own treatment facilities which discharge effluent into the Decatur sewer system. The particular mixture of domestic, industrial, and partially treated wastes received at the Decatur plant is conducive to the formation of a poor settling filamentous sludge. Filamentous conditions have been a historical problem with and continue to seriously plague the air aerated trains. According to plant personnel, filamentous infestation is much less prevalent in the oxygen sludge, but is present in sufficient quantities that a substantially less dense settled sludge is produced than predicted. Even so, oxygen clarifier underflow concentrations range from 70-100 percent higher than comparable data for settled air sludge.

The inability to thicken oxygen sludge during clarification to the degree planned has resulted in lower MLSS and higher F/M operating conditions than designed for. These conditions have apparently not adversely affected effluent quality which remained good throughout the first year of operation, as indicated in Table 8. The somewhat higher effluent suspended solids value shown for February corresponded to an average influent flow equal to 115 percent of design. The effluent data given in Table 8 represented UNOX system effluent quality prior to mixing with air system effluent or subsequent treatment in the tertiary lagoons.

Recent communication with the assistant plant manager elicited the following observations on his part:

1. The oxygen system has consistently outperformed the air system by

TABLE 8. OPERATING AND PERFORMANCE DATA FOR DECATUR, ILLINOIS OXYGEN SYSTEM

Parameter	Design	Operation		
		Aug. 1975	Feb. 1976	July 1976
Influent Flow (mgd)*	17.7	14.4	20.4	14.1
Aeration Detention Time, Q (hr)	1.6	1.97	1.39	2.01
F/M Loading (kg BOD_5/day/kg MLVSS)	0.62	1.10	1.47	0.91
Secondary Clarifier Overflow Rate (gpd/sq ft)†	560	456	645	446
MLSS (mg/l)	5500	2700	3300	2600
Return Sludge TSS (%)	1.9	0.55	0.93	0.87
Reactor Influent BOD_5 (mg/l)	188	157	129	98
TSS (mg/l)	138	139	138	99
Secondary Effluent BOD_5 (mg/l)	20	9	15	10
TSS (mg/l)	25	22	36	20

*1 mgd = 0.044 cu m/sec
†1 gpd/sq ft = 0.041 cu m/day/sq m

a wide margin, despite treating approximately twice as much flow in a substantially smaller reactor volume.

2. The oxygen system has exhibited excellent day-to-day process reliability and is generally capable of recovering from slug loading upsets within 24 hours.

3. Oxygen dissolution and supply systems require more operator attention than conventional air processes, primarily because of the greater amount of instrumentation involved. Several equipment malfunctions to date have been beyond the ability of the plant operating staff to correct and have required attention on the part of the vendor.

4. Several PSA compressor outages were experienced during early operations due to an improper inner cooling system. The cooling system was eventually redesigned and rebuilt and is now performing satisfactorily.

The upgrading modifications implemented at Decatur have resulted in an increase in plant capacity from 20 mgd (0.9 cu m/sec) to 25 mgd (1.1 cu m/sec) and a substantial improvement in total plant performance. Two-thirds of the upgraded 25 mgd (1.1 cu m/sec) capacity is assigned to the new oxygen system, one-third to the existing air system.

Detroit (#1), Michigan (UNOX)

Initial planning for expansion to secondary treatment at Detroit called for the installation of a 1200 mgd (52.6 cu m/sec) air-activated sludge facility to be completed over a four-phase construction period spanning approximately ten years. Two 150-mgd (6.6-cu m/sec) air train modules were to be installed during each construction phase, yielding an eventual total of eight modules.

Coinciding with Detroit's planning program, the use of oxygen in the activated sludge process was being investigated in a federally supported research project at Batavia, New York (1)(2)(3). Based primarily on promising results emanating from this project, Detroit became interested in utilizing oxygen in its own treatment situation. The City made a decision in 1969 to modify its first construction phase to include one 150-mgd (6.6-cu m/sec) air module and one 300-mgd (13.1-cu m/sec) UNOX module. The reactor tanks for both systems were designed with identical outside dimensions, meaning that the aeration detention time of the oxygen system was to be one-half of that of the air system. The high-rate treatment potential of the oxygenation process was of utmost importance to the City because of a serious land shortage problem.

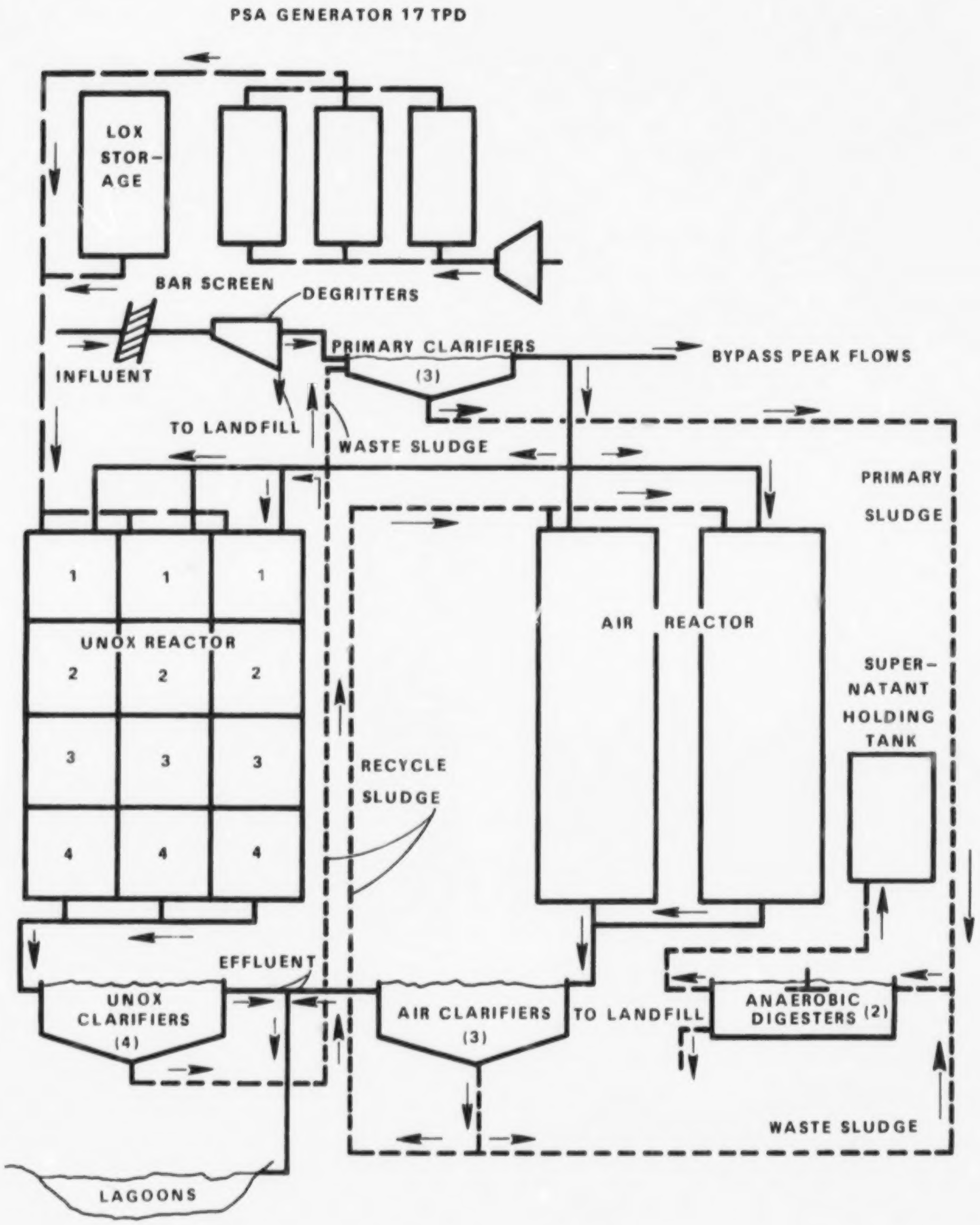

PSA GENERATOR 17 TPD
LOX STOR-AGE
BAR SCREEN
DEGRITTERS
INFLUENT
PRIMARY CLARIFIERS (3)
BYPASS PEAK FLOWS
TO LANDFILL
WASTE SLUDGE
PRIMARY SLUDGE
1
1
1
UNOX REACTOR
2
2
2
3
3
3
4
4
4
AIR REACTOR
SUPER-NATANT HOLDING TANK
RECYCLE SLUDGE
EFFLUENT
UNOX CLARIFIERS (4)
AIR CLARIFIERS (3)
TO LANDFILL
ANAEROBIC DIGESTERS (2)
WASTE SLUDGE
LAGOONS

Figure 6. Flow diagram of Decatur, Illinois wastewater treatment plant. (Printed, with modifications, through the courtesy of the Union Carbide Corporation.)

The construction contract awarded by the City included process guarantee requirements for the UNOX system in the areas of effluent quality, power consumption, and oxygen consumption. The efficacy of the UNOX and air systems was to be compared in parallel test runs. Depending on the results of the tests, the two systems were designed such that the 150-mgd (6.6-cu m/sec) air train could be readily converted to a 300-mgd (13.1-cu m/sec) oxygen train by the addition of a tank cover, submerged turbine/sparger units for oxygen dissolution, and three more secondary clarifiers. With this possibility in mind, the compressors which continuously recirculate gas through the submerged turbine/sparger units were double sized to handle 600 mgd (26.3 cu m/sec). If the test results indicated superior performance by the air train, the City retained the option by virtue of the identical reactor designs of switching the higher capacity oxygen system to a 150-mgd (6.6-cu m/sec) air system by removing the tank cover and substituting air draft tubes for the oxygen dissolution equipment.

For the test runs, six secondary clarifiers were to be mated with the oxygen reactor, three with the air reactor. The clarifiers are of unique design with a diameter of 200 ft (61 m), a SWD of 16 ft (4.9 m), an extremely high average surface overflow rate of 1600 gpd/sq ft (65 cu m/day/sq m), rapid sludge removal suction pipes, and a peripheral-feed rim-takeoff flow configuration. Although the oxygen module has been in operation since August 1974, the writer is not aware of the publication of any officially conducted comparative test results on the two systems to date. Normal start-up problems and delays in getting nine clarifiers completed reportedly contributed to the delay in parallel testing. Whether official test data are eventually published or not, it would appear that Detroit is committed to oxygen use. Two new 300-mgd (13.1-cu m/sec) oxygen modules are now under construction as part of the City's second-phase construction program. The second-phase oxygen systems will utilize OASES equipment. If Detroit decides at a future date to convert the air train installed under first-phase construction to oxygen service, the City will have realized its ultimate goal of 1200 mgd (52.6 cu m/sec) of treatment capacity with four reactor modules instead of eight. A flow diagram for the first-phase air and oxygen modules is given in Figure 7.

The large 30-ft (9.1-m) reactor SWD employed in first-phase construction necessitated the use of the submerged turbine oxygen dissolution alternative. The overall dimensions of the UNOX reactor are 600 ft long x 140 ft wide x 33 ft deep (183 m x 42.7 m x 10.1 m). Oxygen gas is supplied to the UNOX system by a 180-ton/day (163-metric ton/day) cryogenic generator. A 900-ton (816-metric ton) liquid oxygen storage tank provides backup.

Following start-up, it became evident that sufficient detail had not been given to the design of the submerged turbine assemblies. Propeller failures and gear box problems resulted from inadequate materials selection and fabrication. Redesign and partial equipment replacement were necessary to correct the deficiencies. Another problem encountered by the plant staff was obtaining a tight seal at the joints between the outside edges of the reactor cover and the reactor walls. Despite experiments with several different sealants and sealing procedures, this situation was only marginally rectified at the time of this writing. Cryogenic generator performance has been very satisfactory with minimal downtime. During the first 550 days of operation, less than 2.5 percent scheduled and 0.4 percent unscheduled outages were experienced.

Average operating and performance data for the UNOX system are documented in Table 9 for September 1975 and a 1-1/2 month period in the spring of 1976. These data were generated at constant influent flow. Imposition of diurnal flow variations on the UNOX system will be postponed until the remainder of the secondary treatment trains under construction come on-line. It is obvious that reactor influent BOD_5 concentrations have been considerably lower than expected. The weaker strength primary effluent is partially attributable to the recent initiation of iron addition to the primary clarifiers for phosphorus removal.

Two major operational problems have surfaced with the secondary clarifiers. One involves achieving proper peripheral influent distribution to avoid short circuiting of mixed liquor solids directly up to the rim-takeoff weirs. The other is the extreme difficulty encountered in getting settled sludge to thicken to acceptable concentrations prior to removal from the

TABLE 9. OPERATING AND PERFORMANCE DATA FOR DETROIT (#1), MICHIGAN OXYGEN SYSTEM

Parameter	Design	Operation 1975	Operation March 29 - May 9, 1976
Influent Flow (mgd)*	300	302	299
Aeration Detention Time, Q (hr)	1.42	1.41	1.42
F/M Loading (kg BOD_5/day/kg MLVSS)	0.47	0.58	1.05
Secondary Clarified Overflow Rate (gpd/sq ft)†	1600	1611	1595
MLSS (mg/l)	6250	2340	2750
Return Sludge TSS (%)	--	0.66	0.85
Reactor Influent BOD_5 (mg/l)	140	44	101
TSS (mg/l)	150	105	240
Secondary Effluent BOD_5 (mg/l)	25	6	17
TSS (mg/l)	30	9	31

*1 mgd = 0.044 cu m/sec
†1 gpd/sq ft = 0.041 cu m/day/sq m

clarifiers. The impact of the thin settled sludge situation is evident in Table 9 in low MLSS levels and high F/M loadings. The Detroit oxygen sludge does have good thickening properties as exemplified by the ability to separately thicken waste sludge to 4 percent solids in 24 hours without chemical conditioners. In the writer's opinion, secondary clarifier operational difficulties will continue as long as the clarifiers are subjected to the inordinately high overflow rates currently in use.

Fairfax County, Virginia (OASES)

The Westgate plant is one of four municipal wastewater treatment plants operated by Fairfax County, Virginia. This plant, constructed in 1954, was originally designed to remove 50 percent of the BOD_5 loading from a design flow of 8 mgd (0.35 cu m/sec).

Basic features of the original Westgate facility in sequence consisted of bar screening, comminution, primary clarification, once-through aeration, secondary clarification, and chlorination. Sludge recycle pumps were not provided. The main treatment basin was divided into two parallel tanks. Each tank housed primary clarification, aeration and secondary clarification sections separated only by baffles. Scraper chains passed along the entire floor length through all the sections of the tanks. The apparent purpose of the scrapers was to move biological solids and grit settling out in the secondary clarification zones back to the primary clarification zone, where they could be removed from the system. Although the original plant was not intended to function as an activated sludge system, it is highly likely that some settled solids were resuspended in the aeration zones during scraping transport, thus maintaining a small active biomass population in those zones. The decision to forego installation of the additional clarifier appurtenances, sludge recycle equipment, and piping which would have permitted operation in a conventional activated sludge mode was necessitated by funding limitations at the time of initial construction.

From 1954 to 1965, plant influent flows increased gradually from 8 mgd (0.35 cu m/sec) to slightly less than 10 mgd (0.44 cu m/sec). BOD_5 and suspended solids removals during this period averaged approximately 50 and 65 percent, respectively. By 1970 with plant flows having further increased to approximately 11 mgd (0.48 cu m/sec, BOD_5 removal had dropped to 45 percent and suspended solids removal to 55 percent. In 1970, faced with the choice of either upgrading BOD_5 removal efficiency to 80 percent or having a building moratorium placed on the area served by the plant, the County submitted a report to the State of Virginia recommending that interim upgrading steps be applied at Westgate pending com-

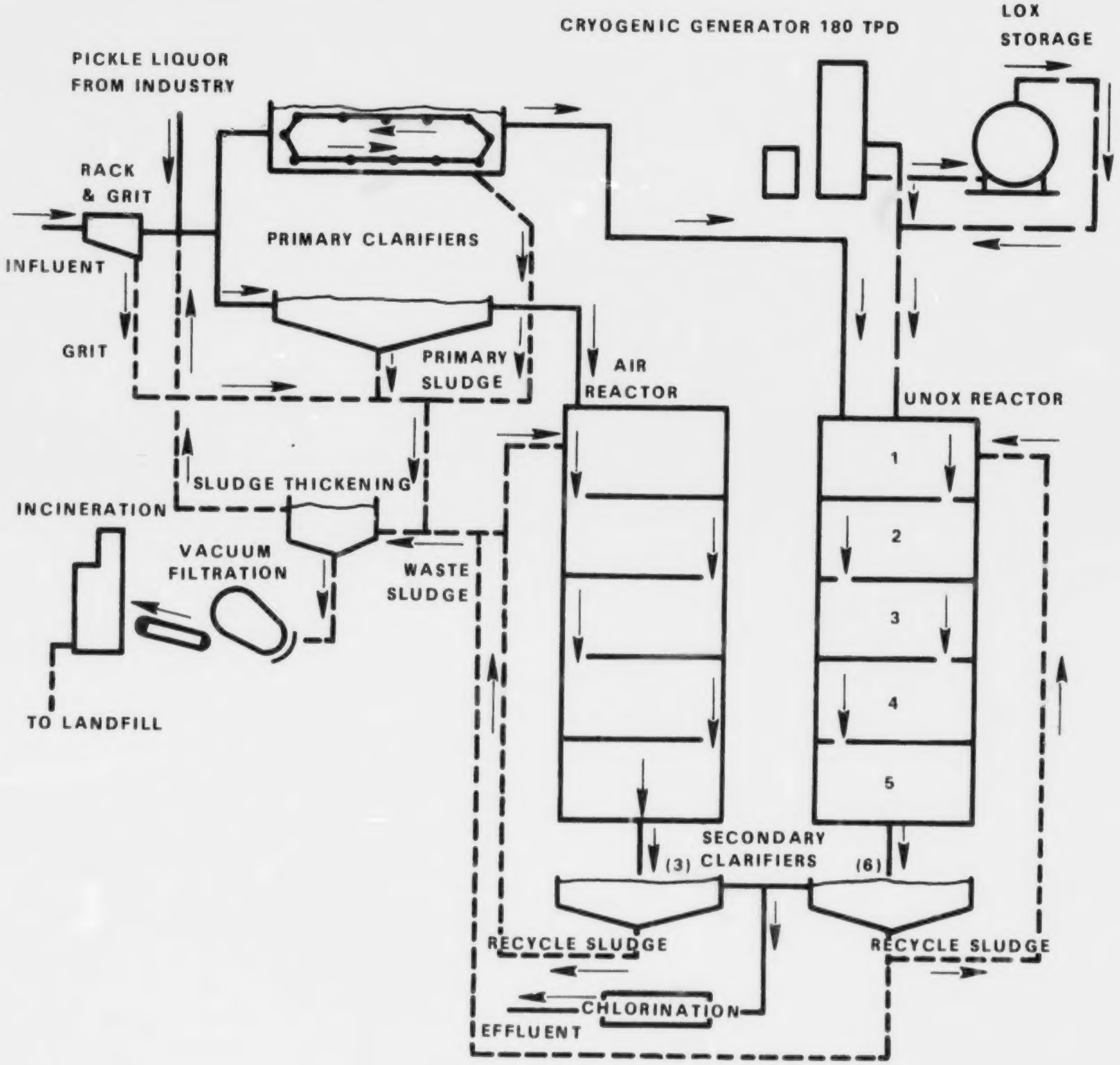

CRYOGENIC GENERATOR 180 TPD
LOX STORAGE
PICKLE LIQUOR FROM INDUSTRY
RACK & GRIT
INFLUENT
PRIMARY CLARIFIERS
GRIT
PRIMARY SLUDGE
AIR REACTOR
UNOX REACTOR
1
2
3
4
5
SLUDGE THICKENING
INCINERATION
VACUUM FILTRATION
WASTE SLUDGE
TO LANDFILL
SECONDARY CLARIFIERS
(3)
(6)
RECYCLE SLUDGE
RECYCLE SLUDGE
CHLORINATION
EFFLUENT

Figure 7. Flow diagram of Detroit, Michigan wastewater treatment plant - Phase #1 construction. (Printed, with modifications, through the courtesy of the Union Carbide Corporation.)

pletion of an expansion program at the nearby Alexandria, Virginia plant. At that time, the Westgate facility would cease operations in favor of flow diversion to Alexandria.

The first interim upgrading approach tried was the addition of ferric chloride to the influent wastewater at the plant headworks followed by anionic polyelectrolyte addition to the aeration zones. This technique yielded an average BOD_5 removal of 71 percent from July 1970 through October 1971, somewhat short of 80 percent removal target. The sludge resulting from chemical addition proved to be more difficult to dewater than that of the original plant.

Laboratory tests indicated that combining powdered activated carbon dosing to the influent wastewater with the above iron and polyelectrolyte additions could potentially improve BOD_5 removal to 75 percent. Full-scale trials with carbon dosing were abandoned in July 1971 after a short-term run due to erosion and feed control problems. Data generated during the run were inconclusive.

During the latter portion of 1970, the County and its engineering consultant concluded that 80 percent interim BOD_5 removal could be achieved more cost effectively with a biological treatment system than with a combination of chemical addition procedures. A decision was then made following technical deliberations to implement biological treatment with an oxygen-activated sludge process rather than a high-rate air-activated sludge process because of reliability and cost considerations. A contract was awarded to Air Products and Chemicals, Inc. in the spring of 1971 after competitive bidding to convert the existing Westgate plant to an OASES system. A contract period of 210 days was allowed to complete the job.

The upgrading plan developed by the County's engineer consisted of four principal steps:

1. Conversion of the aeration and secondary clarification sections of the existing tanks into a two-train oxygenation reactor leaving the primary clarification sections intact.

2. Installation of two new secondary clarifiers, each 120 ft (36.6 m) in diameter with a SWD of 11 ft (3.4 m) and suction lift scraper arms for removing sludge.

3. Installation of waste activated sludge thickening capability in the form of two flotation thickeners, each having a surface area of 250 sq ft (23.2 sq m).

4. Installation of two 7-mgd (0.31-cu m/sec sludge recycle pumps and separate sludge wasting pumps.

A longitudinal section view of the existing Westgate treatment basin prior to conversion to an oxygen system is given in Figure 8. Some of the modifications required to effect the conversion are noted. These included removal of the air diffusers and downcomer piping, removal of the baffles between the old aeration and secondary clarification sections, removal of all old effluent weir sections within the secondary clarification sections proper, removal of the old sludge scrapers from the aeration and secondary clarification zones, replacement of the baffles separating the primary clarification and original aeration zones, and relocation of some sludge scraper sprockets to the primary clarification sections. The converted oxygen reactor was divided into four stages in each train. The stages comprised in order 22, 44, 23, and 11 percent of the total reactor volume. Only the first three stages were covered, the last stage being left open to the atmosphere because of the low oxygen demand which would exist at that point. The gas-tight tank covers and liquid staging baffles were fabricated from carbon steel and coated with an epoxy-phenolic resin. The overall dimensions of the converted oxygen reactor are 138 ft long x 82 ft wide x 12 ft SWD (42 m x 25 m x 3.7 m).

A total of 36 surface aerators with bottom impellers were installed for oxygen dissolution and mixing. Eight of the aerators (utilized at the front end of the reactor) are 10-hp (7.5-kw) units; the other 28 have 5-hp (3.7-kw) drives, yielding a total installed nameplate power load of 220 hp (164 kw). Liquid oxygen is stored on-site and vaporized preceding introduction to the oxygenation system.

Plant modifications were completed and the converted system started up in

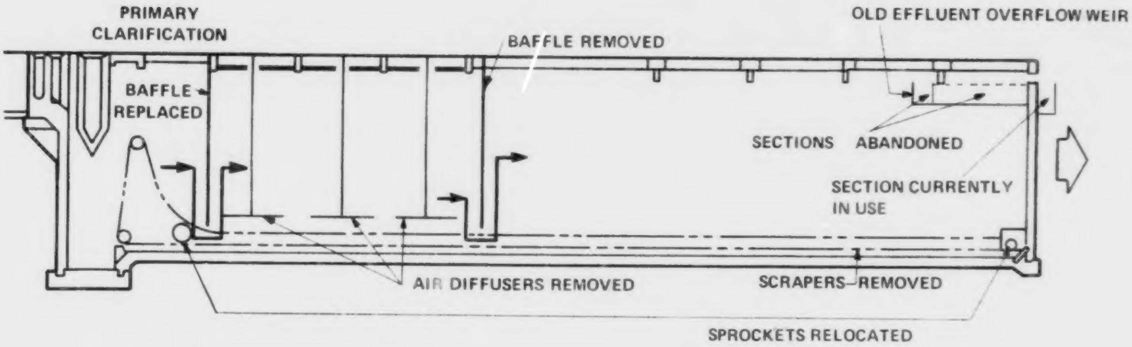


Figure 8. Longitudinal sectional view of pre-modified concrete tank at Fairfax County (Westgate), Virginia wastewater treatment plant. (Reprinted from draft report for EPA Contract No. 68-03-0405.)(6)

November 1971, making Westgate the oldest full-scale oxygen activated sludge facility in the world.

A flow diagram of the modified plant is shown in Figure 9. Operation of the new flotation thickeners was terminated after several months. It was found that thickening of excess activated sludge beyond that afforded by gravity decant tanks was not needed prior to mixing with primary sludge and vacuum filtration. The comminutor was also removed from service several months into the upgraded operation.

Start-up difficulties were minimal and of the type normally associated with "debugging" a new system. A process optimization program was undertaken for the County by Air Products and Chemicals from late January 1972 to May 1972. The primary purpose of the program was to define the operating conditions for this first-of-a-kind system which would result in a consistently high level of plant performance. Operating and performance data are presented in Table 10 for the one-year period of August 1972 through July 1973. As indicated, excellent effluent quality was achieved, far exceeding the 80 percent BOD_5 removal design specification, at an average influent flow equal to 76 percent of design capacity. Primary influent rather than reactor influent concentrations are included in Table 10 because representative sampling of primary effluent is not possible. Little alteration of influent wastewater characteristics is believed to be effected by the primaries due to their short detention time (20-25 minutes).

The Westgate story is a superb example of utilizing existing tankage to the fullest in an upgrading project intended to simultaneously improve plant performance and increase plant capacity. It is not known in view of the excellent performance achieved to date whether the upgraded

TABLE 10. OPERATING AND PERFORMANCE DATA FOR FAIRFAX COUNTY, VIRGINIA OXYGEN SYSTEM

Parameter	Design	Operation Aug. 1972-July 1973
Influent Flow (mgd)*	14	10.6
Aeration Detention Time, Q (hr)	1.74	2.3
F/M Loading (kg BOD_5/day/kg MLVSS)	--	0.54§
Secondary Clarifier Overflow Rate (gpd/sq ft)†	620	469
MLSS (mg/l)	--	4480
Return Sludge TSS (%)	--	1.87
Primary Influent (mg/l)‡		
BOD_5	220	161
TSS	173	162
Secondary Effluent (mg/l)		
BOD_5	44	12
TSS	--	19

*1 mgd = 0.044 cu m/sec

†1 gpd/sq ft = 0.041 cu m/day/sq m

‡Not possible to sample reactor influent as only a baffle wall separates primary clarifier from reactor.

§Based on primary influent BOD_5 rather than reactor influent BOD_5; indicated value is, therefore, about 10 percent high.

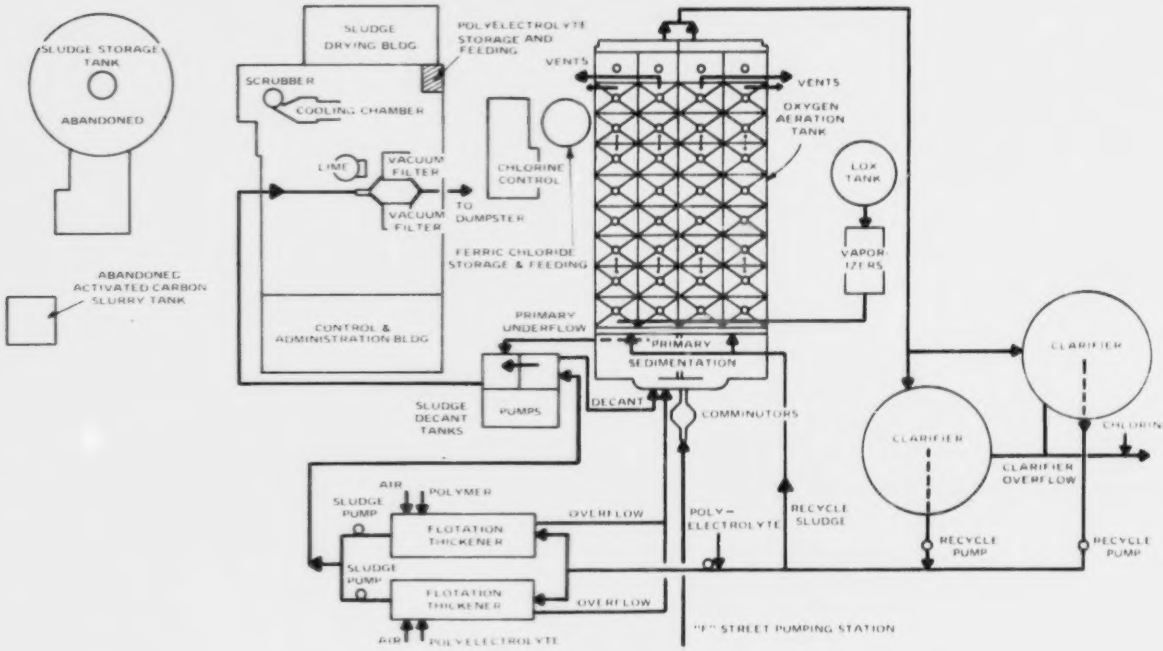

SLUDGE STORAGE TANK
ABANDONED
ABANDONED ACTIVATED CARBON SLURRY TANK
SLUDGE DRYING BLDG
POLYELECTROLYTE STORAGE AND FEEDING
SCRUBBER
COOLING CHAMBER
LIME
VACUUM FILTER
VACUUM FILTER
TO DUMPSTER
CHLORINE CONTROL
FERRIC CHLORIDE STORAGE & FEEDING
CONTROL & ADMINISTRATION BLDG
PRIMARY UNDERFLOW
SLUDGE DECANT TANKS
PUMPS
DECANT
VENTS
VENTS
OXYGEN AERATION TANK
LOX TANK
VAPORIZERS
PRIMARY SEDIMENTATION
COMMINUTORS
CLARIFIER
CLARIFIER
CHLORINE
CLARIFIER OVERFLOW
RECYCLE PUMP
RECYCLE PUMP
AIR
POLYMER
SLUDGE PUMP
FLOTATION THICKENER
OVERFLOW
SLUDGE PUMP
FLOTATION THICKENER
OVERFLOW
AIR
POLYELECTROLYTE
POLY-ELECTROLYTE
RECYCLE SLUDGE
"F" STREET PUMPING STATION

Figure 9. Flow diagram of Fairfax County (Westgate), Virginia wastewater treatment plant. (Reprinted from draft report for EPA Contract No. 68-03-0405.)(6)

plant will still be abandoned when the Alexandria expansion is completed or not. The total cost of the Westgate upgrading was $1,672,000, of which $861,000 was expended for the oxygen dissolution and supply systems and reactor tank modifications.

Gulf States Paper Corporation, Tuscaloosa, Alabama (UNOX)

A custom-designed, self-contained, circular UNOX system was installed at the Gulf States Paper Corporation complex in Tuscaloosa, Alabama, to treat 9 mgd (0.39 cu m/sec) of unbleached kraft mill wastewater. This type of wastewater is deficient in nitrogen and phosphorus. To overcome these deficiencies at Gulf States, phosphoric acid and anhydrous ammonia are added to the primary effluent.

A custom-designed circular UNOX system differs from one of Union Carbide's modular package oxygen plants in that it is not a standard off-the-shelf unit. The Gulf States oxygen system is composed of three above-ground steel tanks each with a diameter of 109 ft (33.2 m), a total depth of 20 ft (6.1 m), and a SWD of 16 ft (4.9 m). Each tank is divided into a four-stage oxygenation reactor and an arcuate clarifier. Three of the four stages are also arcuate; one is circular. Air-lift suction pickups are used to withdraw settled sludge from the clarifiers. Oxygen dissolution and solids mixing are accomplished with surface aerators and bottom propellers. A four-bed 30-ton/day (27.2-metric ton/day) PSA oxygen gas generator and a 43-ton (39-metric ton) liquid oxygen backup storage tank and atmospheric vaporizer comprise the oxygen supply system.

As shown in the flow diagram presented in Figure 10, alum can be dosed to a separate polishing clarifier following secondary clarification for the purpose of effecting additional color removal. This color removal system has not been used to any great extent to date, however, because of problems with the alum recovery equipment.

The oxygen system itself has been in operation since October 1974. Following start-up and "debugging," maintenance requirements have been of a routine nature. Operator attention on the unit ranges from 7-10 hr/week.

Operating and performance data for the months of April and May 1975 are summarized in Table 11. Although the system is operating at design flow, reactor influent strength has been averaging only about 60 percent of design expectations. It has, therefore, not been necessary to operate at as high MLSS levels as projected to maintain reasonable F/M loadings. The effluent values shown represent product quality from the secondary clarifiers. Additional suspended solids removal is reportedly achieved

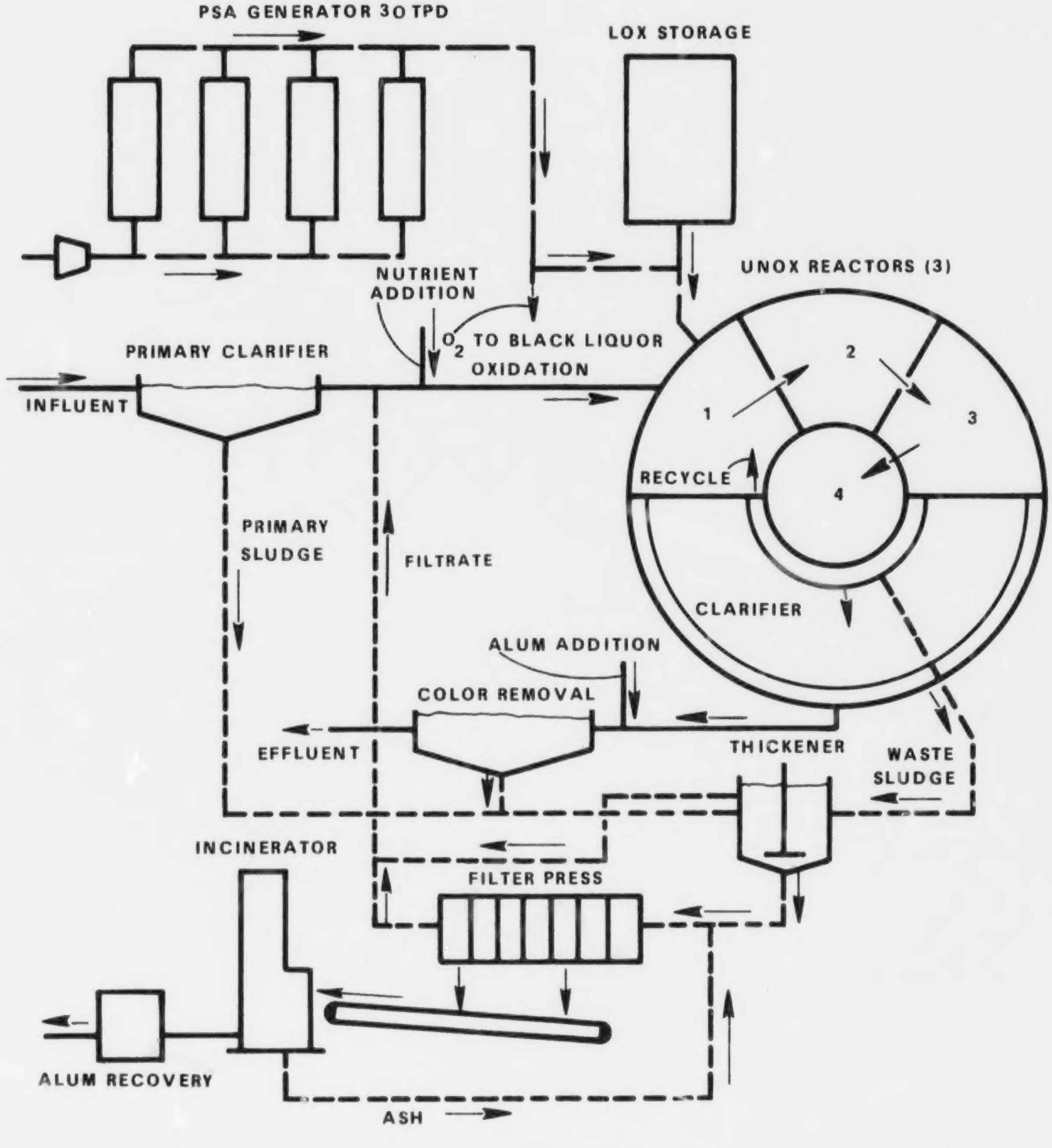

PSA GENERATOR 30 TPD
LOX STORAGE
NUTRIENT ADDITION
UNOX REACTORS (3)
O_2 TO BLACK LIQUOR OXIDATION
PRIMARY CLARIFIER
INFLUENT
1
2
3
4
RECYCLE
PRIMARY SLUDGE
FILTRATE
CLARIFIER
ALUM ADDITION
COLOR REMOVAL
EFFLUENT
THICKENER
WASTE SLUDGE
INCINERATOR
FILTER PRESS
ALUM RECOVERY
ASH

Figure 10. Flow diagram of the Gulf States Paper Corporation wastewater treatment plant - Tuscaloosa, Alabama. (Printed, with modifications, through the courtesy of the Union Carbide Corporation.)

in passage through the polishing clarifier (operated without alum addition). No data were available to the writer to document the improvement obtained in the polishing clarifier.

Approximately one-half of the PSA generator output is used in the activated sludge system; the other half is utilized for black liquor oxidation. Because of the dual role served by the oxygen supply facilities, the PSA unit was designed to produce 95 percent purity oxygen gas rather than the standard 90 percent product purity normally associated with PSA operation.

TABLE 11. OPERATING AND PERFORMANCE DATA FOR GULF STATES PAPER OXYGEN SYSTEM

Parameter	Design	Operation Apr. - May 1975
Influent Flow (mgd)*	9.0	9.0
Aeration Detention Time, Q (hr)	3.33	3.33
F/M Loading (kg BOD_5/day/kg MLVSS)	0.36	0.37
Secondary Clarifier Overflow Rate (gpd/sq ft)†	630	630
MLSS (mg/l)	4700	3000
Return Sludge TSS (%)	1.9	1.2
Reactor Influent (mg/l)		
BOD_5	200	125
TSS	100	60
Secondary Effluent (mg/l)		
BOD_5	30	12
TSS	50	50

*1 mgd = 0.044 cu m/sec
†1 gpd/sq ft = 0.041 cu m/day/sq m

Lederle Laboratories, Pearl River, New York (UNOX)

Lederle Laboratories, a division of American Cyanamid, manufactures pharmaceuticals, the majority of which are antibiotics. The waste stream resulting from production operations has a very high and variable organic carbon content. The plant wastewater flow which remains relatively constant at about 1.0 mgd (0.044 cu m/sec) can have a BOD_5 loading as high as 32,500 lb/day (14,740 kg/day).

Prior to the spring of 1972, an air aeration system was used to treat plant wastes. The daily operations of this system were marked by persistent odor problems and inconsistent performance, arising from the highly variable organic load. A UNOX system was designed to replace the existing air aeration facilities. Start-up occurred in March 1972, which makes it the oldest permanent full-scale UNOX facility in existence.

A flow diagram of the new oxygenation treatment plant is given in Figure 11. The two-train reactor has overall dimensions of 148 ft long x 74 ft wide x 14.5 ft deep (45 m x 22.6 m x 4.4 m) with a SWD of 10 ft (3.0 m). The lead reactor stages are larger than the second or third stages to accommodate the high oxygen demand of the incoming wastewater. Polymers are added ahead of the single primary clariflocculator to lower the suspended solids concentration entering the secondary system as much as possible. The three circular secondary clarifiers each have a 40-ft (12.2-m) diameter, a 10-ft (3.0-m) SWD, and a plow-type sludge scraper. A 15-ton/day (13.6-metric ton/day) PSA generator and a 52-ton (47-metric ton) liquid oxygen back-up tank provide oxygen supply.

Start-up difficulties included a foaming tendency which ceased once a good biomass had been established, and mixed liquor solids deposition caused by recycle of large amounts of lime and alum precipitates in the filtrate from the vacuum filter which are not effectively captured in the primary clariflocculator. Solids deposition was alleviated by adding bottom mixers to the initially supplied surface aerators. The PSA oxygen generator experienced upwards of 10 percent outage following start-up due to valve actuator problems. This unit was one of the first on-line molecular sieve applications geared to producing oxygen gas for wastewater treatment. As such, some experimentation was necessary to determine proper lubricating procedures for the valve actuators and to procure sufficiently rugged valve equipment to withstand rapid cycling. Following final modifications in mid-1973, total unscheduled PSA generator downtime has been reduced to less than one percent.

Odor complaints from neighboring residents numbered more than 80 in 1971. Complaints have not been received since the oxygen system went into operation.

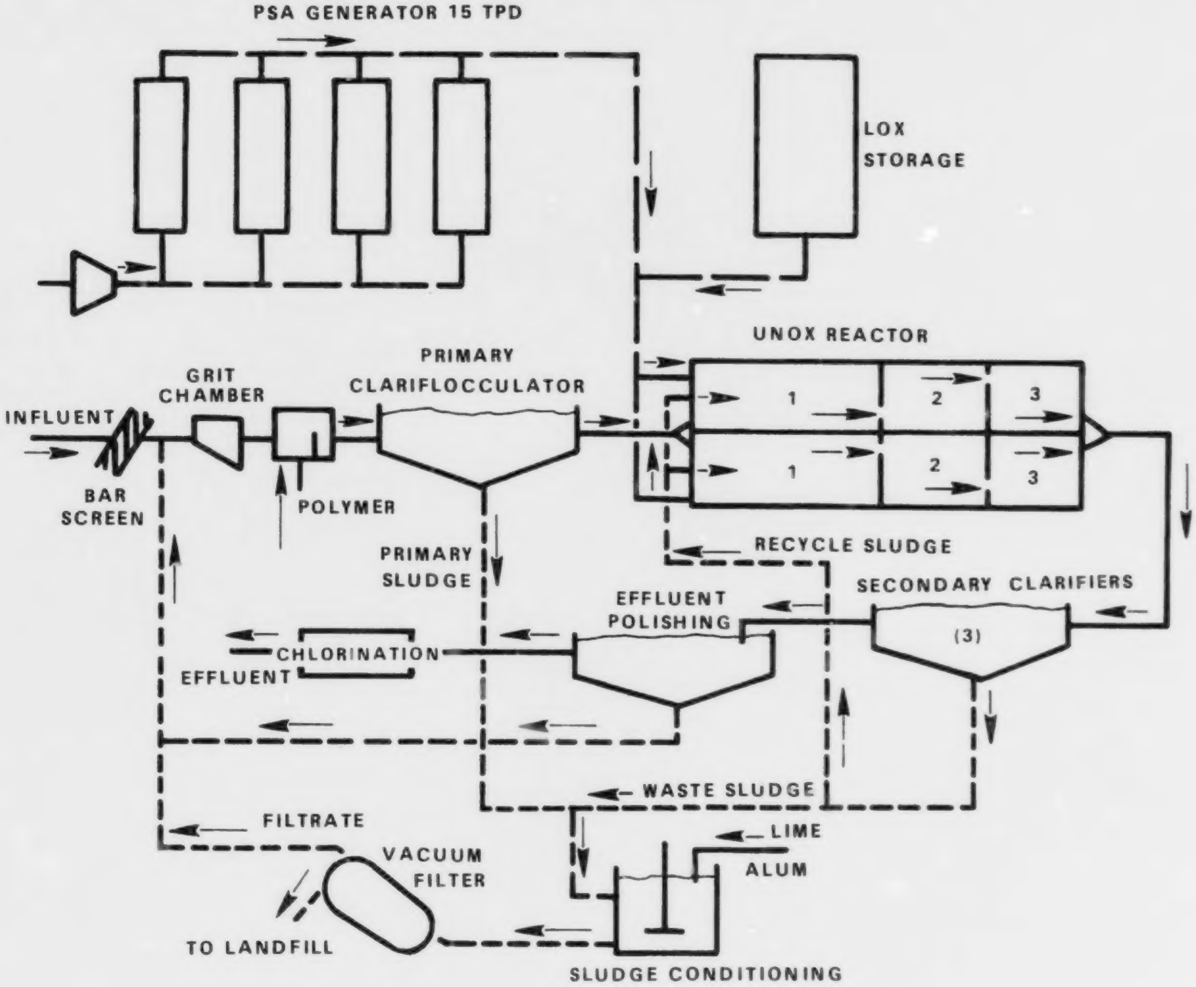

PSA GENERATOR 15 TPD
LOX
STORAGE
UNOX REACTOR
PRIMARY
CLARIFLOCCULATOR
GRIT
CHAMBER
INFLUENT
BAR
SCREEN
POLYMER
1
2
3
1
2
3
RECYCLE SLUDGE
PRIMARY
SLUDGE
SECONDARY CLARIFIERS
(3)
EFFLUENT
POLISHING
CHLORINATION
EFFLUENT
WASTE SLUDGE
FILTRATE
LIME
ALUM
VACUUM
FILTER
TO LANDFILL
SLUDGE CONDITIONING

Figure 11. Flow diagram of Lederle Laboratories wastewater treatment plant - Pearl River, New York. (Printed, with modifications, through the courtesy of the Union Carbide Corporation.)

A 50 percent sludge recycle rate has been necessary to prevent settled activated sludge from thickening to concentrations greater than 3-4 percent in the secondary clarifiers. During a three-month period when the recycle rate was decreased, the clarifier underflow concentration increased to 5-6 percent. Sludge pumping problems ensued at the higher concentrations, necessitating a return to the 50 percent rate.

Experienced flows have remained approximately one-third less than design. This has afforded Lederle the opportunity to remove one reactor train from service during summer months when the biochemical reaction rate is at its highest level. The partial shutdown action is taken to conserve energy during the time plant manufacturing energy requirements are greatest. During the remainder of the year, both trains are kept in service to minimize excess sludge production through operation at lower F/M loadings. None of the three secondary clarifiers are taken out of operation except for maintenance.

Average performance data for back-to-back test periods in 1972 are summarized in Table 12 for both one-train and two-train operation. Since the 1972 test period, sludge removal problems in the secondary

TABLE 12. OPERATING AND PERFORMANCE DATA
FOR LEDERLE LABORATORIES OXYGEN SYSTEM

Parameter	Design	Operation	
		2 Trains Oct. 1972	1 Train Nov. 1972 (3 weeks)
Influent Flow (mgd)*	1.5	1.0	1.0
Aeration Detention Time, Q (hr)	13	19.5	9.75
F/M Loading (kg BOD_5/day/kg MLVSS)	0.42	0.17	0.45
Secondary Clarifier Overflow Rate (gpd/sq ft)†	540	360	360
MLSS (mg/l)	8000	11,500	9600
Return Sludge TSS (%)	2.8	3.5	3.0
Reactor Influent BOD_5 (mg/l)	1600	1400	1500
TSS (mg/l)	--	800	1300
Polishing Clarifier Effluent BOD_5 (mg/l)	160	80	90
TSS (mg/l)	--	70	60

*1 mgd = 0.044 cu m/sec
†1 gpd/sq ft = 0.041 cu m/day/sq m

clarifiers have been corrected. Settled sludge no longer fills up the secondaries and spills over into the polishing clarifier. With the polishing clarifier serving in its intended role, effluent BOD_5 and suspended solids concentrations now generally average around 50 and 10 mg/l, respectively.

Littleton, Colorado (UNOX)

The City of Littleton, Colorado, selected a modular UNOX system for a recent plant expansion. The modular unit used for Littleton is an off-the-shelf package system contained within one circular aboveground steel tank. The tank, 82 ft (25 m) in diameter x 15 ft (4.6 m) deep [SWD = 12 ft (3.7 m)], is divided by internal walls into a two-stage oxygen reactor, an arcuate secondary clarifier, a single-stage air aerobic sludge digester, and a chlorine contact chamber. The arcuate clarifier is equipped with floating bridge mounted air lift suction equipment for withdrawing settled sludge.

The new oxygen train operates in parallel with two existing trickling filters. Feed to the trickling filters is first settled in the plant's existing primary clarifier. The oxygen reactor receives raw degritted municipal wastewater directly. A flow diagram for the Littleton plant is presented in Figure 12.

The combined liquid volume of the two oxygen reactor stages is 97,000 gal (367 cu m). Surface aerators connected by shafts to bottom propellers are employed for oxygen dissolution and mixing. Due to the small size of the treatment plant, an on-site oxygen gas generating facility was not provided. Instead, liquid oxygen is trucked in and stored in a 43-ton (39-metric ton) tank, from where it is directed through an atmospheric vaporizer for conversion to the gaseous form before entering the oxygenation reactor.

The UNOX package system became operational in February 1974. A major operating difficulty was immediately encountered. The original uncovered air aerobic sludge digester was equipped with mechanical surface aerators. Aerator icing occurred in the cold Colorado winter climate with resulting poor volatile suspended solids (VSS) reduction. The problem was rectified by installing a steel cover over the digester area along with urethane foam insulation and supplementing the surface aerators with an air blower and diffusers to provide adequate air circulation. VSS reductions have since ranged from 50-60 percent.

Influent flow to the oxygen portion of the Littleton plant has varied from 0.9 mgd

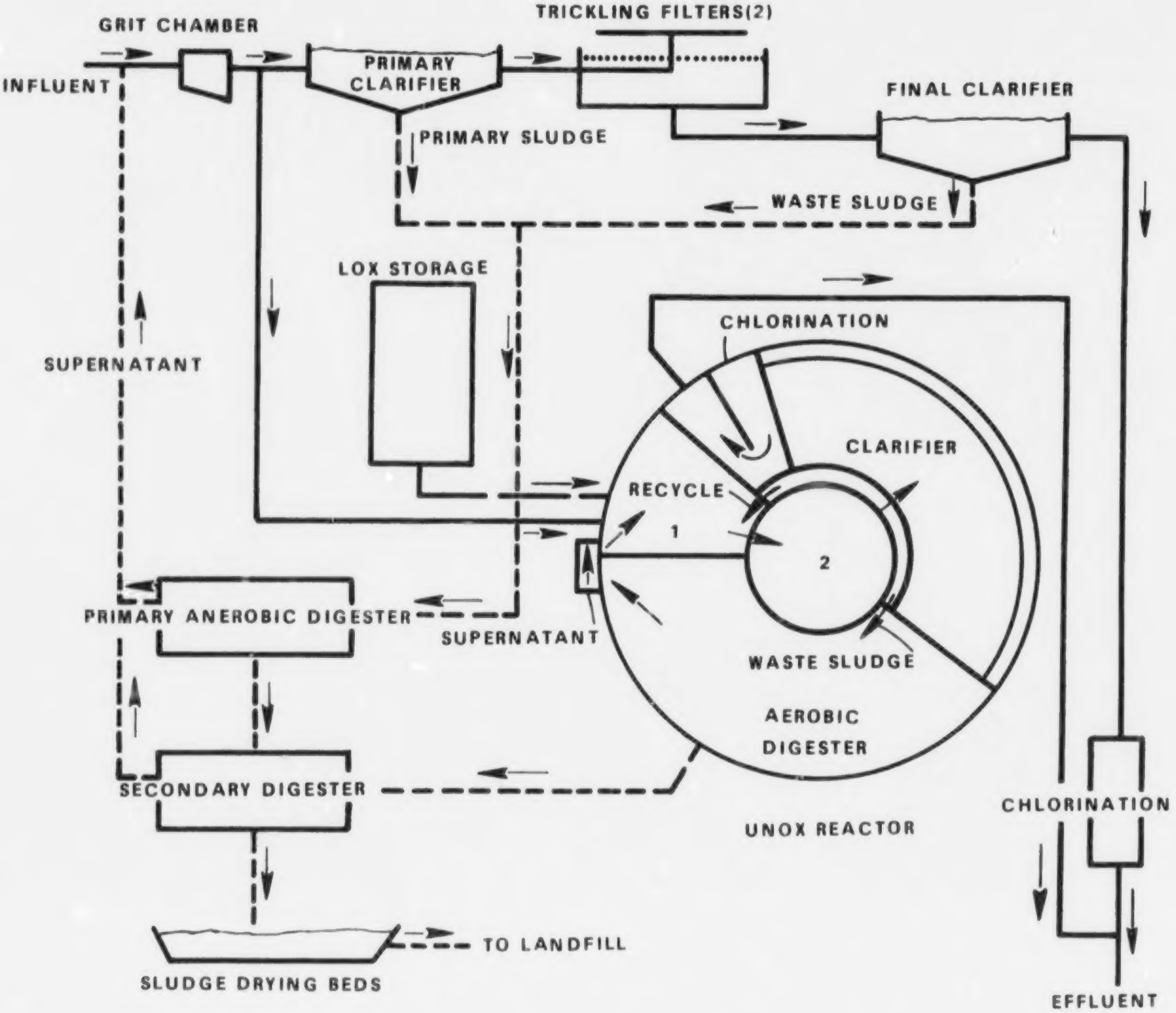

GRIT CHAMBER
TRICKLING FILTERS(2)
INFLUENT
PRIMARY CLARIFIER
FINAL CLARIFIER
PRIMARY SLUDGE
WASTE SLUDGE
LOX STORAGE
CHLORINATION
SUPERNATANT
CLARIFIER
RECYCLE
1
2
PRIMARY ANEROBIC DIGESTER
SUPERNATANT
WASTE SLUDGE
AEROBIC DIGESTER
SECONDARY DIGESTER
CHLORINATION
UNOX REACTOR
TO LANDFILL
SLUDGE DRYING BEDS
EFFLUENT

Figure 12. Flow diagram of Littleton, Colorado wastewater treatment plant. (Printed, with modifications, through the courtesy of the Union Carbide Corporation.)

(0.04 cu m/sec) to 1.4 mgd (0.06 cu m/sec) since start-up. The three-month average data summarized in Table 13 for the summer 1975 period indicate the oxygen system is performing within effluent design specifications.

Morganton, North Carolina (UNOX)

A UNOX facility designed to treat 8 mgd (0.35 cu m/sec) of municipal wastewater combined with substantial industrial contributions resulting from textiles production and poultry processing went on-stream at Morganton, North Carolina, in January 1975. As indicated in the plant flow diagram (Figure 13), primary clarification was not included in the design. The two-train oxygen reactor was constructed in an unusual box configuration with four stages per train. Each stage is 44 ft (13.4 m) square yielding overall length and width dimensions of 88 ft (26.8 m) and 176 ft (53.6 m), respectively. The total reactor depth is 14 ft (4.3 m) including a 4-ft. (1.2-m) freeboard.

Oxygen system equipment consists of surface aerators with bottom impellers for oxygen dissolution and mixing and a 26-ton/

TABLE 13. OPERATING AND PERFORMANCE DATA FOR LITTLETON, COLORADO OXYGEN SYSTEM

Parameter	Design	Operation June-Aug. 1975
Influent Flow (mgd)*	1.25	1.1
Aeration Detention Time, Q (hr)	1.9	2.16
F/M Loading (kg BOD_5/day/kg MLVSS)	0.6	0.6
Secondary Clarifier Overflow Rate (gpd/sq ft)†	500	440
MLSS (mg/l)	5600	4000
Return Sludge TSS (%)	2.8	2.4
Reactor Influent (mg/l)		
BOD_5	200	160
TSS	240	185
Secondary Effluent (mg/l)		
BOD_5	20	12
TSS	25	24

*1 mgd = 0.044 cu m/sec
†1 gpd/sq ft = 0.041 cu m/day/sq m

day (23.6-metric ton/day) PSA generator and 28-ton (25.4-metric ton) liquid oxygen backup tank for oxygen supply. The PSA generator was outfitted initially with one one-half size compressor. A second half-size compressor will be added at a later date when plant flows increase. The two new secondary clarifiers are 80-ft (24.4-m) diameter units with 10-ft (3.0-m) SWD's and rapid sludge removal and grease skimming capabilities.

Process and mechanical reliability have been excellent in the year and half since start-up. PSA generator availability has exceeded 99.5 percent. A major operational problem in the form of high fat and grease loadings (often in excess of 100 mg/l) from the local poultry processor, however, has prevented consistent attainment of effluent quality objectives. No satisfactory method exists for rejecting these objectionable materials from the secondary system. The fat and grease which are only slowly biodegradable pass to the final clarifiers and collect on the liquid surfaces. Although skimming devices were provided, much of the scum escapes the finals over the weirs taking with it significant quantities of enmeshed biofloc. Consequently, effluent suspended solids have reached levels as high as 80-100 mg/l. The problem is further accentuated by a hydraulic regime which transports final clarifier skimmings to the aerobic digesters and then recycles the digester skimmings to the plant headworks for recycle through the secondary system.

Efforts to remove a large fraction of the fat and grease load through pretreatment at the poultry processing site have been unsuccessful. Consideration is now being given to intercepting the skimmings from the aerobic digesters and disposing of them separately. When this technique has been evaluated for short periods on a trial basis, effluent clarity and suspended solids removals have improved measurably. The difficulties encountered at Morganton appear to the writer to consitute a compelling argument for the inclusion of primary clarification facilities in any future designs faced with similar wastewater characteristics.

Two months of operating and performance data are summarized in Table 14. Effluent quality documented for March 1975 is typical of months when the influent grease load has been somewhat lower than normal and represents about the best performance level that can be achieved under present conditions. In April 1975, the influent grease load was up, and the monthly average effluent suspended solids concentration increased accordingly. The much higher than anticipated influent suspended solids concentrations for these two months indicate that primary clarification would probably have been a desirable and justifiable feature aside from grease removal considerations.

North Lauderdale, Florida (UNOX)

An off-the-shelf modular UNOX system was installed at North Lauderdale, Florida, to serve a population base of approximately 10,000 people. This package system consists of a two-stage oxygen reactor, an arcuate secondary clarifier, and an uncovered single-stage air aerobic sludge digester. The arcuate clarifier has an air lift suction device mounted from a floating bridge for removing settled sludge. The air aerobic digester is equipped only with mechanical surface aerators: it was not necessary to provide supplemental compressed air as at Littleton.

The entire secondary complex is contained within one circular, 95-ft (29-m) dia-

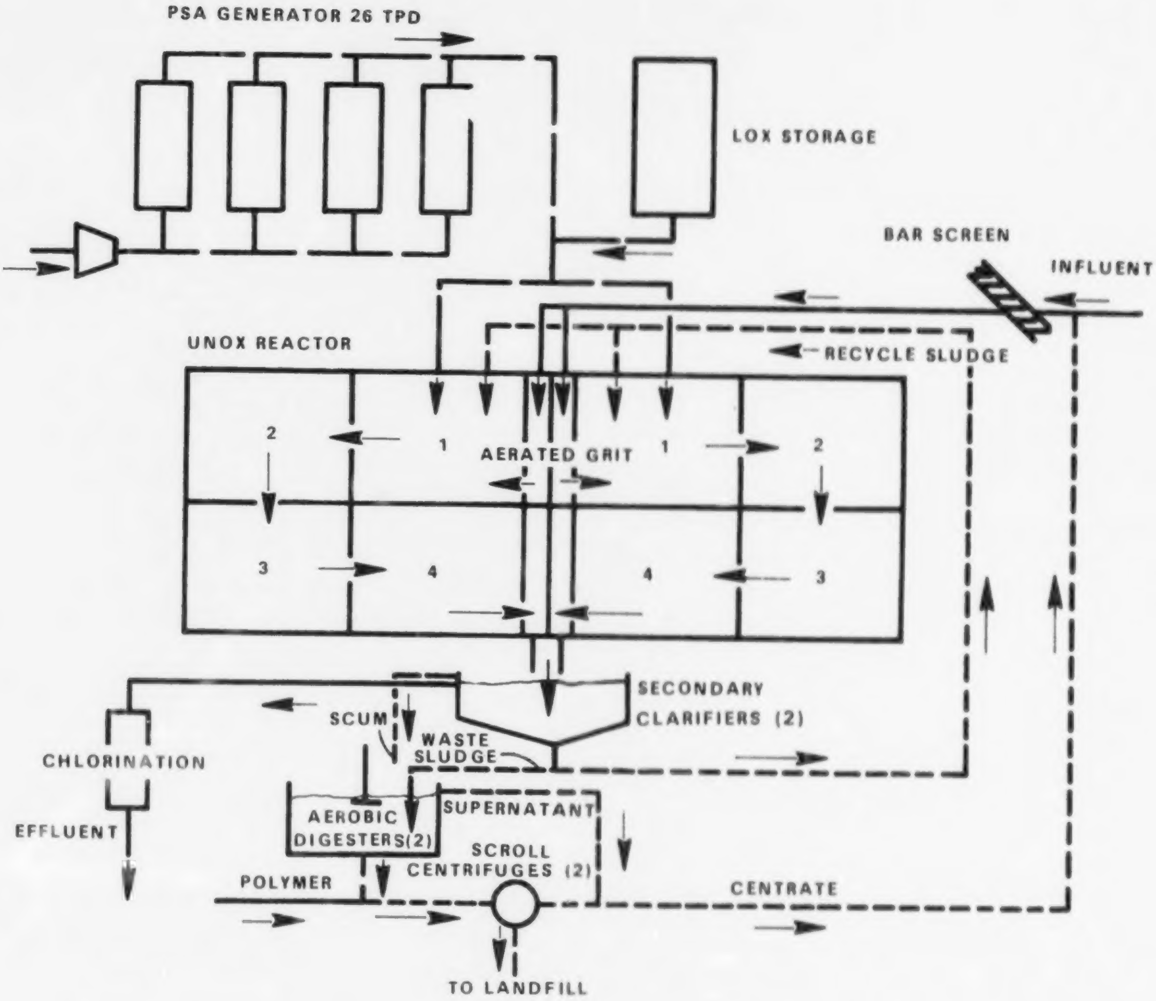

PSA GENERATOR 26 TPD
LOX STORAGE
BAR SCREEN
INFLUENT
UNOX REACTOR
RECYCLE SLUDGE
2
1
AERATED GRIT
1
2
3
4
4
3
SECONDARY CLARIFIERS (2)
SCUM
WASTE SLUDGE
CHLORINATION
EFFLUENT
AEROBIC DIGESTERS(2)
SUPERNATANT
SCROLL CENTRIFUGES (2)
POLYMER
CENTRATE
TO LANDFILL

Figure 13. Flow diagram of Morganton, North Carolina wastewater treatment plant. (Printed, with modifications, through the courtesy of the Union Carbide Corporation.)

meter, 15-ft (4.6-m) deep, above-ground steel tank. The tank's SWD is 12 ft (3.7 m). Unlike the Littleton modular unit, a separate external chlorine contact chamber was provided rather than including it in the package system. Granular media filters are available for effluent polishing, although to date they have not been used. Raw degritted municipal wastewater is fed directly to the oxygen system. Excess activated sludge is dewatered either by centrifugation or on sand drying beds. A flow diagram of the new North Lauderdale treatment plant is shown in Figure 14.

Oxygen dissolution and mixing are accomplished in the 129,000-gal (488-cu m) oxygen reactor with surface aerators and supplemental bottom agitators. A 43-ton (39-metric ton) liquid oxygen storage tank and attendant atmospheric vaporizer comprise the oxygen supply system.

The plant was placed in operation in early July 1975. To date, influent flow has been averaging only about 65 percent of the design flow of 2 mgd (0.09 cu m/sec), although wastewater strength has been somewhat higher than anticipated. No significant operating problems have been encountered. Performance as exhibited by the average data for September 1975 shown in Table 15 has been excellent.

TABLE 14. OPERATING AND PERFORMANCE DATA FOR MORGANTON, NORTH CAROLINA OXYGEN SYSTEM

Parameter	Design	Operation March 1975	Operation April 1975
Influent Flow (mgd)*	8.0	4.7	5.9
Aeration Detention Time, Q (hr)	3.5	6.0	4.7
F/M Loading (kg BOD_5/day/kg MLVSS	0.53	0.31	0.33
Secondary Clarifier Overflow Rate (gpd/sq ft)†	800	470	590
MLSS (mg/l)	6000	5600	6400
Return Sludge TSS (%)	3.0	2.1	1.6
Reactor Influent BOD_5 (mg/l)	350	364	357
TSS (mg/l)	400	836	946
Secondary Effluent BOD_5 (mg/l)	27	42	32
TSS (mg/l)	25	29	79

*1 mgd = 0.044 cu m/sec
†1 gpd/sq ft = 0.041 cu m/day/sq m

TABLE 15. OPERATING AND PERFORMANCE DATA FOR NORTH LAUDERDALE, FLORIDA OXYGEN SYSTEM

Parameter	Design	Operation Sept. 1975
Influent Flow (mgd)*	2.0	1.3
Aeration Detention Time, Q (hr)	1.56	2.4
F/M Loading (kg BOD_5/day/kg MLVSS)	0.68	0.68
Seconday Clarifier Overflow Rate (gpd/sq ft)†	525	541
MLSS (mg/l)	5600	4500
Return Sludge TSS (%)	2	1.5‡
Reactor Influent (mg/l)		
BOD_5	200	245
TSS	130	180
Secondary Effluent (mg/l)		
BOD_5	20	<10
TSS	20	<10

*1 mgd = 0.044 cu m/sec
†1 gpd/sq ft = 0.041 cu m/day/sq m
‡High return sludge rate

Speedway, Indiana (UNOX)

In June 1972, a new 7.5 mgd (0.33 cu m/sec) UNOX installation was placed in operation at Speedway, Indiana. This was the first municipal UNOX facility to be completed. Of all the oxygen-activated sludge wastewater treatment systems now in operation, the Speedway plant was preceded only by the municipal OASES plant at Fairfax County, Virginia, and the industrial UNOX plant at the Lederle Laboratories in Pearl River, New York.

The flow diagram in Figure 15 indicates that the Speedway oxygen system is of conventional design. The two-train oxygen reactor is preceded by primary clarification. Three of the six primaries are existing units; the other three are converted secondary clarifiers from the City's old trickling filter treatment facility. Each of the four reactor stages per train is 22 ft (6.7 m) square with a 16-ft (4.9-m) SWD. The overall dimensions of the two reactor tanks taken together are 88 ft long x 44 ft wide x 20 ft deep (26.8 m x 13.4 m x 6.1 m). Three new 65-ft (19.8-m) diameter, 10-ft (3.0-m) SWD secondary clarifiers with the increasingly popular rapid method of removing settled sludge were provided. At a future date as needed, plant capacity can be increased to 10 mgd (0.44 cu m/sec) by the construction of one additional secondary clarifier.

The UNOX reactors were designed to use surface aerators attached by shafts to bottom agitators for oxygen dissolution and mixing. A three-bed 5-ton/day (4.4-metric ton/day) PSA unit generates oxygen gas on-site. A 7-ton (6.4-metric ton) liquid oxygen storage tank and accompanying atmospheric vaporizer were furnished for reserve. Profiting from difficulties experienced with earlier four-bed PSA gen-

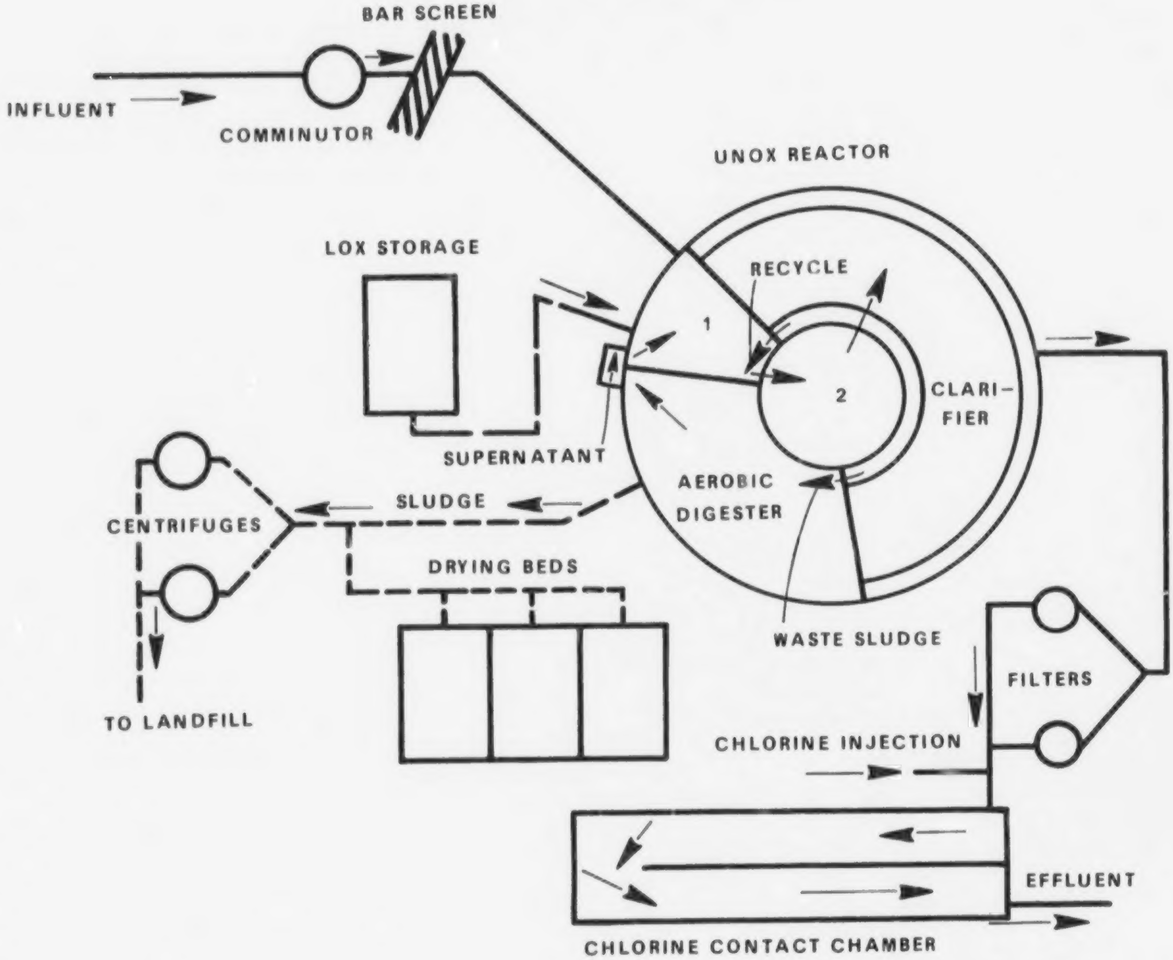

BAR SCREEN
INFLUENT
COMMINUTOR
UNOX REACTOR
LOX STORAGE
RECYCLE
1
2
CLARI-FIER
SUPERNATANT
AEROBIC DIGESTER
SLUDGE
CENTRIFUGES
DRYING BEDS
WASTE SLUDGE
FILTERS
TO LANDFILL
CHLORINE INJECTION
EFFLUENT
CHLORINE CONTACT CHAMBER

Figure 14. Flow diagram of North Lauderdale, Florida wastewater treatment plant. (Printed, with modifications, through the courtesy of the Union Carbide Corporation.)

erator designs at Lederle Laboratories and on a U.S. EPA co-sponsored demonstration grant project at the Newtown Creek plant in Brooklyn, New York (3), particularly as related to valves and lubricants, the second generation three-bed design employed at Speedway has proven to be highly reliable with less than 1-1/2 percent total downtime for scheduled and unscheduled maintenance.

Waste activated sludge is recycled to the primaries for co-thickening with raw sludge. The mixed sludges are then pumped to a holding tank which feeds a Zimpro wet oxidation system designed to condition sludge for dewatering. Conditioned sludge is dewatered by vacuum filtration prior to being trucked to landfill. Periodic and lengthy shutdowns of the wet oxidation system placed considerable stress on the main stream treatment components for much of the early history of this new facility. Unable to truck liquid sludges away, it was frequently necessary to return mixed raw and waste sludges from the sludge holding tank to the primary clarifiers. When the primaries filled up, sludge overflowed into the oxygen reactors along with primary effluent. The oxygenation tanks during these periods in effect served more as aerobic sludge digesters than conventional activated sludge systems.

Considering the difficulties imposed by the above conditions on the management of

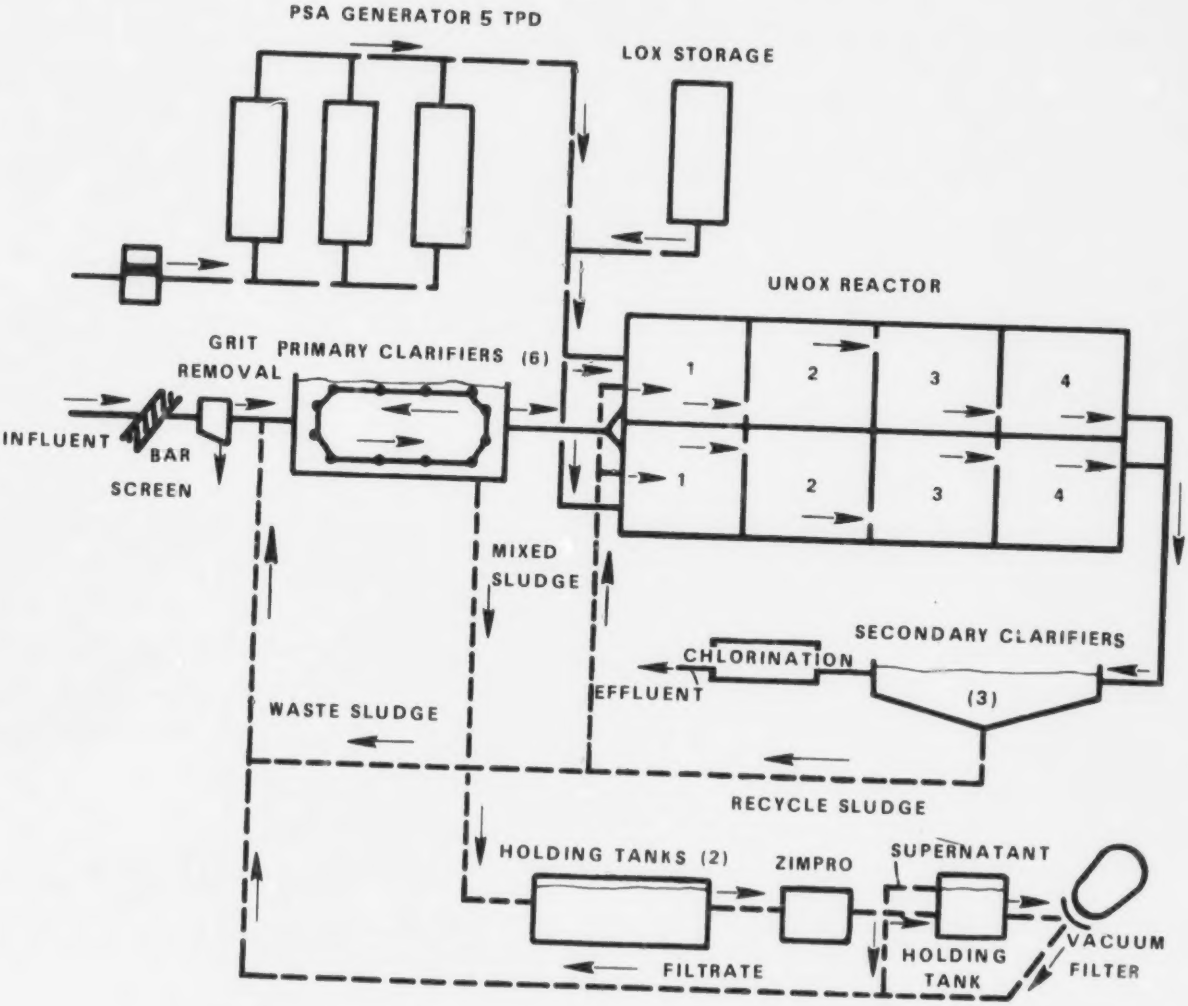

PSA GENERATOR 5 TPD
LOX STORAGE
UNOX REACTOR
GRIT REMOVAL
PRIMARY CLARIFIERS (6)
INFLUENT
BAR SCREEN
1
2
3
4
1
2
3
4
MIXED SLUDGE
SECONDARY CLARIFIERS
CHLORINATION
EFFLUENT
(3)
WASTE SLUDGE
RECYCLE SLUDGE
HOLDING TANKS (2)
ZIMPRO
SUPERNATANT
VACUUM FILTER
HOLDING TANK
FILTRATE

Figure 15. Flow diagram of Speedway, Indiana wastewater treatment plant. (Printed, with modifications, through the courtesy of the Union Carbide Corporation.)

secondary sludge inventory, oxygen system performance was superb. Annual average effluent BOD_5 and suspended solids concentrations were low in both 1973 and 1974, as indicated in Table 16. The highest monthly average BOD_5 and suspended solids levels recorded in these two years were 16 and 30 mg/l, respectively. Also shown in Table 16 are the results of one month of one-train operation in early 1976. Occasional one-train operating tests have been conducted by plant personnel to evaluate oxygen system performance at loadings comparable to design values.

Union Carbide Corporation, Sistersville, West Virginia (UNOX)

The Chemicals and Plastics Division of the Union Carbide Corporation placed a UNOX system in operation in November 1973 to treat waste products from the manufacture of silicones. The resulting wastewater stream has a high organic carbon content and also contains substantial quantities of acid and various oils. Conditioning is necessary ahead of the biological process to neutralize the acid and remove the oil. A holding pond (not included in the flow

TABLE 16. OPERATING AND PERFORMANCE DATA
FOR SPEEDWAY, INDIANA OXYGEN SYSTEM

Parameter	Design	Operation		
		2 Trains[*]		1 Train[†]
		1973	1974	Jan. 16 - Feb. 18, 1976
Influent Flow (mgd)[‡]	7.5	4.4	4.6	4.3
Aeration Detention Time, Q (hr)	1.48	2.52	2.41	1.29
F/M Loading (kg BOD_5/day/kg MLVSS)	0.51	0.20	0.51	0.70
Secondary Clarifier Overflow Rate (gpd/sq ft)[§]	750	440	460	645
MLSS (mg/l)	4200	6080	6600	4760
Return Sludge TSS (mg/l)	2.2	1.54	1.3	1.66[α]
Reactor Influent BOD_5 (mg/l)	110	91	73	114
TSS (mg/l)	96	179	109	118
Secondary Effluent BOD_5 (mg/l)	15	9	9	13
TSS (mg/l)	20	16	14	18

[*] Three secondary clarifiers in operation
[†] Two secondary clarifiers in operation
[‡] 1 mgd = 0.044 cu m/sec
[§] 1 gpd/sq ft = 0.041 cu m/day/sq m
[α] Excludes reported values for Feb. 6, 7, 8, and 9

diagram shown in Figure 16) is utilized for diversion of large spills that cannot be adequately preconditioned.

This inhouse Carbide project marked the first utilization of circular reactor/clarifier UNOX combination tanks. Two such units were installed, each consisting of three arcuate reactor stages and one circular reactor stage and an arcuate final clarifier. In contrast to the above-ground designs employed in later circular UNOX facilities (refer to Gulf Stages Paper Corporation; Littleton, Colorado; and North Lauderdale, Florida), the Sistersville tanks were installed in conventional below-ground fashion. The custom-designed dimensions of the Sistersville units are: diameter - 102 ft (31.1 m), total depth - 14 ft (4.3 m), and SWD - 10 ft (3.1 m). The final clarifiers are equipped with air-lift suction equipment for removing settled sludge.

Surface aerators connected to bottom impellers are used to achieve oxygen dissolution and oxygen and biomass dispersion. Oxygen is supplied in a rather unusual manner from an on-site industrial cryogenic nitrogen gas generator which produces 15 tons/day (13.6 metric tons/day) of oxygen gas as a by-product. Prior to start-up of the silicones wastewater treatment facility, the by-product oxygen was wasted to the atmosphere.

Problems were initially encountered with the floating bridge mechanism from which the sludge scraping and pickup devices are supported. Corrective action required redesign and relocation of the bridge center support. Later arcuate clarifier designs profited from the Sistersville experiences.

Occasional toxic spills, primarily from copper, have resulted in biological upsets. The oxygen-activated sludge system has usually recovered from these spills within one week. Following an in-plant survey, a program is underway to eliminate copper from plant discharges in concentrations which are toxic to microorganisms.

Average monthly operating and performance data for August and December 1975 are presented in Table 17. At influent loadings equal to 85-95 percent of hydraulic capacity, effluent quality has been significantly better than required by the design specifications. The difficulty in settling silicone fines can be noted in the effluent suspended solids levels which are two to three times the effluent BOD_5 concentrations.

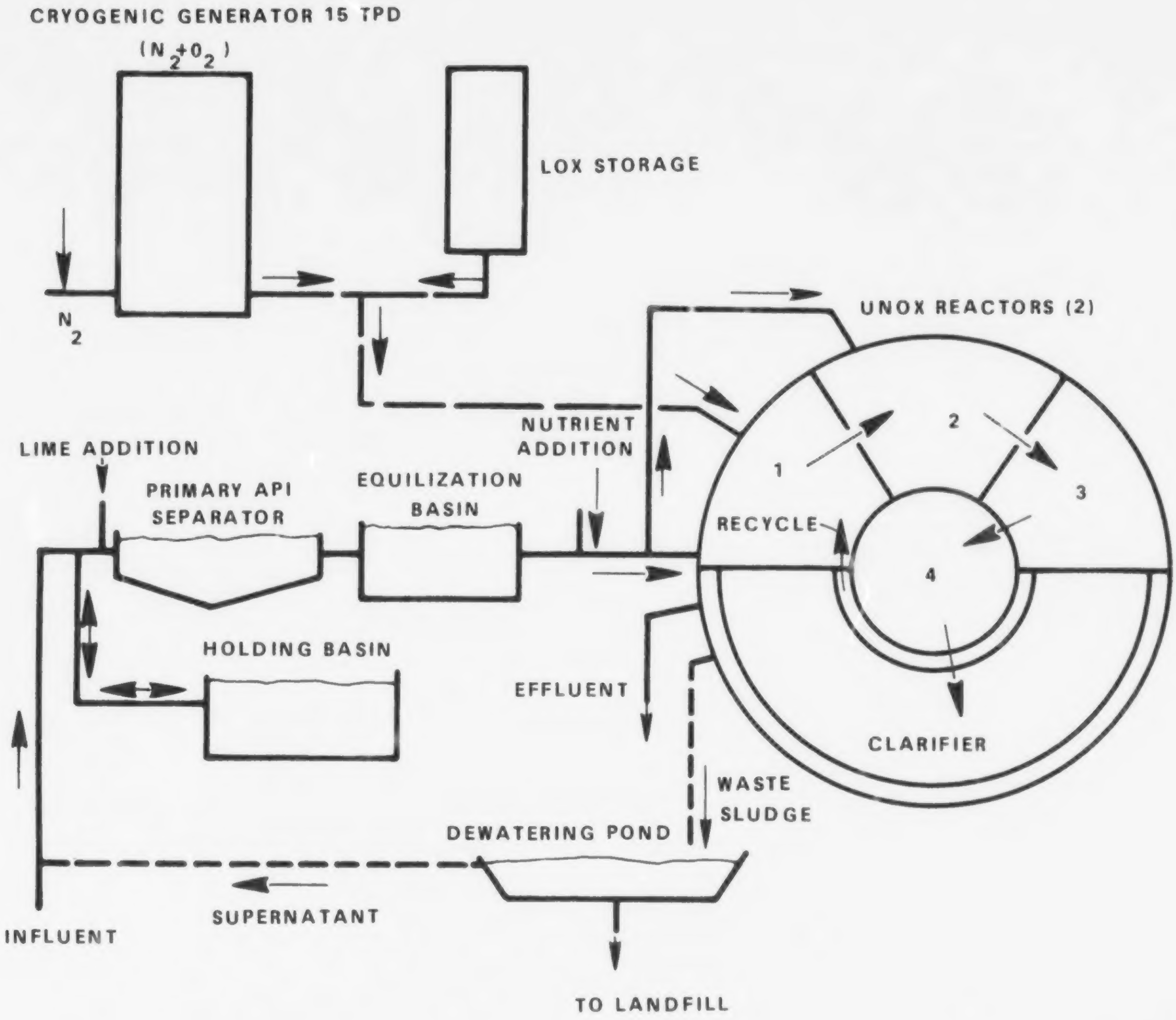

CRYOGENIC GENERATOR 15 TPD
(N_2+O_2)
LOX STORAGE
N_2
UNOX REACTORS (2)
NUTRIENT ADDITION
LIME ADDITION
PRIMARY API SEPARATOR
EQUILIZATION BASIN
1
2
3
4
RECYCLE
HOLDING BASIN
EFFLUENT
CLARIFIER
WASTE SLUDGE
DEWATERING POND
SUPERNATANT
INFLUENT
TO LANDFILL

Figure 16. Flow diagram of Union Carbide Corporation wastewater treatment plant - Sistersville, West Virginia. (Printed, with modifications, through the courtesy of the Union Carbide Corpoation.)

Winnipeg, Manitoba (UNOX)

One of the more attractive oxygen-activated sludge plants now in operation is located at Winnipeg, Manitoba, Canada. This 12-mgd (0.53-cu m/sec) treatment facility was designed to operate over a wide air temperature range (100°F in summer to -50°F in winter) and is, therefore, totally housed with the exception of the covered UNOX reactor.

The plant utilizes a conventional flow scheme to treat municipal wastewater. Primary clarification is utilized ahead of a two-train, three-stage/train, oxygenation reactor having overall dimensions of 120 ft long x 60 ft wide x 19.5 ft deep (36.6 m x 18.3 m x 5.9 m). The reactor's SWD is 16 ft (4.9 m). Mixed liquor flow is evenly divided between two 110-ft (33.5-m) diameter final clarifiers. The SWD of the finals is 10 ft (3.0 m). As with most recently-

TABLE 17. OPERATING AND PERFORMANCE DATA FOR UNION CARBIDE SISTERSVILLE OXYGEN SYSTEM

Parameter	Design	Operation	
		Aug. 1975	Dec. 1975
Influent Flow (mgd)*	4.3	3.6	4.1
Aeration Detention Time, Q (hr)	3.5	4.2	3.7
F/M Loading (kg BOD_5/day/kg MLVSS)	0.85	0.75	0.90
Secondary Clarifier Overflow Rate (gpd/sq ft)†	600	502	572
MLSS (mg/l)	5000	4500	3900
Return Sludge TSS (%)	2.0	1.0‡	2.0
Reactor Influent (mg/l)			
BOD_5	370	425	339
TSS	<100	75	103
Secondary Effluent (mg/l)			
BOD_5	50	25	20
TSS	<100	70	43

* 1 mgd = 0.44 cu m/sec

† 1 gpd/sq ft = 0.041 cu m/day/sq m

‡ High sludge return rate

constructed circular clarifiers, rapid sludge removal equipment was provided rather than the older plow-type scrapers. Primary and waste activated sludges are mixed and centrifuged before undergoing incineration. The flow diagram for the plant is given in Figure 17.

The UNOX system components selected for Winnipeg include surface aerators and bottom mixers for oxygen dissolution and a 10-ton/day (9.1-metric ton/day) PSA oxygen gas generator and 14-ton (12.7-metric ton) liquid oxygen backup tank for oxygen supply. Considerable difficulty has been experienced with the operation of the PSA compressor. This machine was initially outfitted with internal clearance pocket unloaders. These unloaders did not function properly resulting in the imposition of undue stress on and excessive wear of compressor bearings and bushings. Frequent outages were necessary to overhaul the worn parts. Eventually in late 1975, the compressor was completely rebuilt and the clearance pocket unloaders replaced with suction pocket unloaders. Except for one subsequent unscheduled outage due to a heater failure, the compressor has worked well since then.

System start-up occurred in September 1974. No process related difficulties have been encountered. Operation at cold mixed liquor temperatures down to 10°C has not induced growth of filamentous organisms or any other noticeable sludge settling problems. In early 1975, official one-month performance tests were conducted with only one reactor train in service and with both reactor trains in service. Both final clarifiers were used during each test. The results of the tests are documented in Table 18. In each case, although the aeration detention time was less than and the F/M loading higher than design, effluent BOD_5 and suspended solids concentrations were significantly lower than required by design stipulations.

ACKNOWLEDGEMENTS

Information on oxygen systems in various stages of implementation was supplied by the Union Carbide Corporation; Air Products and Chemicals, Inc.; and the FMC Corporation. Case history data and flow diagrams for UNOX plants in operation were provided by the Union Carbide Corporation. Case history data and pertinent diagrams for the OASES plant at Fairfax County, Virginia, were extracted from the final report draft (5) (publication pending) for U. S. EPA Contract No. 68-03-0405.

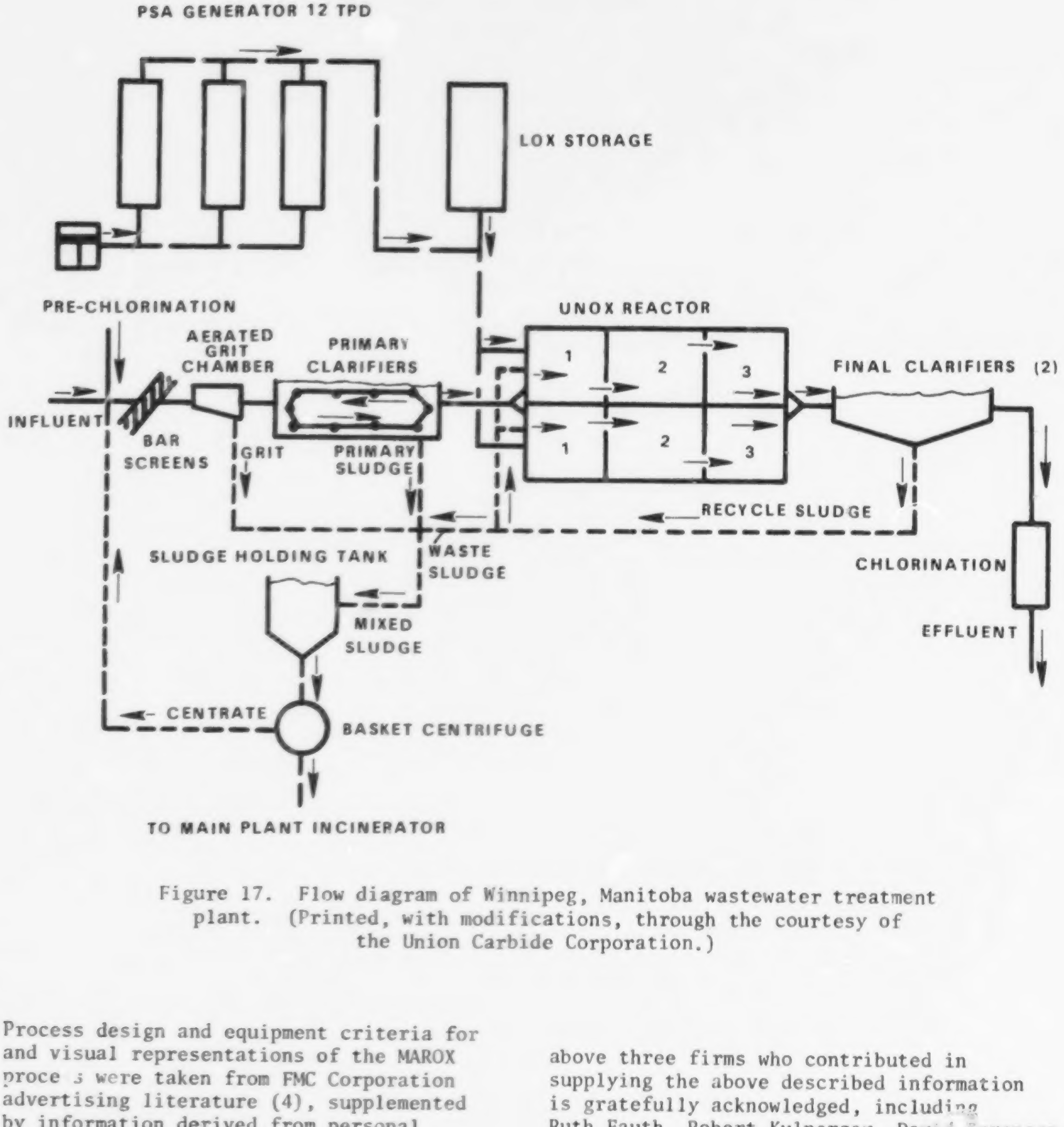

PSA GENERATOR 12 TPD
LOX STORAGE
PRE-CHLORINATION
UNOX REACTOR
AERATED GRIT CHAMBER
PRIMARY CLARIFIERS
1
2
3
1
2
3
FINAL CLARIFIERS (2)
INFLUENT
BAR SCREENS
GRIT
PRIMARY SLUDGE
RECYCLE SLUDGE
SLUDGE HOLDING TANK
WASTE SLUDGE
CHLORINATION
MIXED SLUDGE
EFFLUENT
CENTRATE
BASKET CENTRIFUGE
TO MAIN PLANT INCINERATOR

Figure 17. Flow diagram of Winnipeg, Manitoba wastewater treatment plant. (Printed, with modifications, through the courtesy of the Union Carbide Corporation.)

Process design and equipment criteria for and visual representations of the MAROX process were taken from FMC Corporation advertising literature (4), supplemented by information derived from personal communications with FMC. Progress reports and other information on file at the U. S. EPA's Municipal Environmental Research Laboratory for Grant No. S803910 formed the basis of the discussion of Metropolitan Denver's MAROX demonstration project. Flow and dimensioned diagrams of the Denver MAROX test bay were reprinted from an FMC project bulletin (6).

The assistance of staff members of the above three firms who contributed in supplying the above described information is gratefully acknowledged, including Ruth Fauth, Robert Kulperger, David Sorensen, Michael Lutz, Randy Dievendorf, and Ronald Grader of Union Carbide; O. Roy Langslet of Air Products and Chemicals; and Duane Parker and W. Phillip Key of FMC. Special thanks is extended to Ms. Fauth who spent many days collecting and verifying oxygen plant status data. The cooperation of Richard Kaptain, Assistant Plant Manager for the City of Decatur, Illinois, in providing additional details for the Decatur UNOX case history is also appreciated.

TABLE 18. OPERATING AND PERFORMANCE DATA
FOR WINNIPEG, MANITOBA OXYGEN SYSTEM

Parameter	Design	Operation 1 Reactor Train ‡ Jan. 1975	Operation 2 Reactor Trains ‡ Apr. 1975
Influent Flow (mgd)*	12	8.5	13.4
Aeration Detention Time, Q (hr)	1.74	1.23	1.59
F/M Loading (kg BOD_5/day/kg MLVSS)	0.46	0.99	0.62
Secondary Clarifier Overflow Rate (gpd/sq ft)†	630	446	704
MLSS (mg/l)	5000	5950	5100
Return Sludge TSS (%)	2.2	1.9	1.6
Reactor Influent (mg/l)			
BOD_5	133	244	150
TSS	100	290	193
Secondary Effluent (mg/l)			
BOD_5	25	20	17
TSS	30	17	13

* 1 mgd = 0.044 cu m/sec
† 1 gpd/sq ft = 0.041 cu m/day/sq m
‡ Both final clarifiers in service

REFERENCES

1. Albertsson, J. G., McWhirter, J. R., Robinson, E. K., and Vahldieck, N. P., "Investigation of the Use of High Purity Oxygen Aeration in the Conventional Activated Sludge Process," Water Pollution Control Research Series Report No. 17050 DNW 05/70, Federal Water Quality Administration, Cincinnati, Ohio, May 1970.

2. Brenner, R. C., "Summary Description of Oxygen Aeration Systems in the United States," Proceedings of the Second U. S.-Japan Conference on Sewage Treatment Technology, Cincinnati, Ohio, December 1972.

3. Brenner, R. C., "EPA Experiences in Oxygen-Activated Sludge," Prepared for Office of Technology Transfer Design Seminar Program, U. S. Environmental Protection Agency, Cincinnati, Ohio, October 1974.

4. FMC Corporation, "FMC Pure Oxygen Wastewater Treatment in Open Tanks," FMC Bulletin 8000-A, Itasca, Illinois, 1976.

5. FMC Corporation, "FMC Pure Oxygen System at Metropolitan Denver Sewage Disposal District No. 1," FMC Project Report 8000.1, Itasca, Illinois, 1976.

6. McDowell, C. S., and Giannelli, J., "Oxygen-Activated Sludge Plant Completes Two Years of Successful Operation," Draft Report for Contract No. 68-03-0405 with Air Products and Chemicals, Inc., U. S. Environmental Protection Agency, Cincinnati, Ohio, Publication Pending.

APPENDIX A. OXYGEN-ACTIVATED SLUDGE PLANTS IN OPERATION AS OF JUNE 1976

Location	Design Flow (mgd)*	Installed O_2 Supply Capacity (tons/day)†	Application‡	O_2 Dissolution System§	O_2 Supply System∝
USA					
1. Alton Box Board - Jacksonville, Fla.	6	25	I-PP	UNOX (A)	CRYO
2. Baychem Corp., Chemagro Div. - Kansas City, Kan.	4.32	50	I-C	UNOX (A)	CRYO
3. Brunswick, Ga.	10	16	M	UNOX (A)	PSA
4. Chaska, Minn.	1.25	1.25	M	OASES (A)	LIQ
5. Chesapeake Corp. - West Point, Va.	16.25	34	I-PP	UNOX (A)	CRYO
6. Container Corp. - Fernandino Beach, Fla.	25	50	I-PP(b)	UNOX (A)	CRYO
7. Decatur, Ill.	17	17	M	UNOX (A)	PSA
8. Deer Park, Tex.	5	6	M	UNOX (A)	PSA
9. Denver (#2), Colo.	10	7.5	M	MAROX (R)	LIQ
10. Detroit (#1), Mich.	300	180	M	UNOX (T)	CRYO
11. Fairfax County, Va.	14	10	M	OASES (A)	LIQ
12. Fayetteville, N.C.	14	18	M	OASES (A)	CRYO
13. Fibreboard Corp. - Antioch, Calif	16	35	I-PP(b)	UNOX (A)	PIPE
14. Ft. Myers, Fla.	5	9	M	UNOX (A)	PSA
15. French Paper Co. - Niles, Mich.	0.8	1	I-PP	UNOX (A)	LIQ
16. Gulf States Paper Corp. - Tusaloosa, Ala	10	30	I-PP	UNOX (A)	PSA
17. Hamburg (#1), N. Y.	1	0.5	M	UNOX (A)	LIQ
18. Hercules, Inc. - Wilmington, N.C.	1	15	I-C	UNOX (A)	PIPE
19. Hollywood, Fla	36	50	M	OASES (A)	CRYO
20. Jacksonville (#1), Fla.	5	20	M	UNOX (A)	PSA
21. Lederle Laboratories Div. of American Cyanamid-Pearl River, N.Y.	1.5	15	I-PH	UNOX (A)	PSA
22. Littleton, Colo.	1.5	0.5	M(d)	UNOX (A)	LIQ
23. Morganton, N.C.	8	26	M	UNOX (A)	PSA
24. Morrisville, Pa.	4.6	4	M	UNOX (A)	PSA
25. Newtown Creek - New York City, N.Y.	20	14	M	UNOX (T)	PSA
26. North Lauderdale, Fla.	2	1	M(d)	UNOX (A)	LIQ
27. Quail Valley, Tex.	1.5	2	M	UNOX (A)	PSA
28. Speedway, Ind.	7.5	4	M	UNOX (A)	PSA
29. Standard Brands - Peeksville, N.Y.	1	5	I-FP	UNOX (A)	LIQ
30. Union Carbide Corp. - Marietta, Ga.	1.26	1	I-C	UNOX (A)	LIQ
31. Union Carbide Corp. - Sistersville, W. Va.	4.33	15	I-C	UNOX (A)	PIPE
32. Union Carbide Corp. - Taft, La.	3.8	88	I-PC	UNOX (T)	PIPE
33. Weyerhauser Corp. - Everett, Wash.	3	25	I-PP	UNOX (A)	PIPE
34. Wyandotte, Mich.	100	60	M	UNOX (T)	PSA
35. Yuba City, Calif.	7	21	M	UNOX (A)	PSA
TOTAL	664.61	856.45			
Canada					
1. Winnipeg, Manitoba	12	10	M	UNOX (A)	PSA

APPENDIX A, Continued

	Location	Design Flow (mgd)*	Installed O_2 Supply Capacity (tons/day)†	Application ‡	O_2 Dissolution System§	O_2 Supply System$^{\alpha}$
	Japan					
1.	Electro Chemical Industrial Co. - Ichihara City	2.64	n.d.**	I-PC	UNOX (A)	n.d.**
2.	Gotsu Plant - Katano City	0.73	n.d.	M	UNOX (A)	n.d.
3.	Ikuta Plant -Kawasaki City	0.61	n.d.	M	UNOX (A)	n.d.
4.	Jujo Paper - Kushiro City	1.59	n.d.	I-PP	UNOX (A)	n.d.
5.	Kasuga Plant - Oita City	0.26	n.d.	M	UNOX (A)	n.d.
6.	Mitsubishi Chemical Industries -	1.9	n.d.	I-PC	UNOX (A)	n.d.
7.	Nissho Kayaku Petrochemical Complex - Oita City	0.74	n.d.	I-PC	UNOX (A)	n.d.
8.	Oji Paper - Kasugai City	18.5	n.d.	I-PP	UNOX (A)	n.d.
9.	Oji Paper - Tomakomai City	13.2	n.d.	I-PP	UNOX (A)	n.d.
10.	Sanyo Kakusaku Pulp - Iwakuni City	0.89	n.d.	I-PP	UNOX (A)	n.d.
11.	Showa Neoprene - Kawasaki City	0.79	n.d.	I-SR	UNOX (A)	n.d.
12.	Sumitomo Chemical - Ichihara City	0.79	n.d.	I-PC	UNOX (A)	n.d.
13.	Uenodai Plant, Japan Housing Corp. - Kamifukuoka City	0.52	n.d.	M	UNOX (A)	n.d.
14.	Yakult Pharmaceutical Industries - Osaka City	0.19	n.d.	I-PH	MAROX (R)	n.d.
	TOTAL	43.35				

*1 mgd = 0.044 cu m/sec

†1 ton/day = 0.907 metric ton/day

‡I-C = Industrial-Chemicals
I-FP = Industrial-Food Processing
I-PC = Industrical-Petrochemical
I-PH = Industrial-Pharmaceutical
I-PP = Industrial-Pulp & Paper
I-SR = Industrial-Synthetic Rubber
M = Municipal
(b) - Conventional O_2 treatment plus black liquor oxidation
(d) = Conventional O_2 treatment plus aerobic sludge digestion

§MAROX = FMC Corp.
OASES = Air Products & Chemicals, Inc.
UNOX = Union Carbide Corp.
(A) = Surface aerators (with or without bottom mixers)
(F) = Fixed active diffusers
(R) = Rotating active diffusers
(T) = Submerged turbines

$^{\alpha}$CRYO = On-site liquid oxygen gas generation
LIQ = On-site liquid oxygen storage and vaporization
PIPE = Pipeline transport of oxygen gas from a nearby off-site oxygen generating facility
PSA = On-site pressure swing adsorption oxygen gas generation

**n.d. = no data.

APPENDIX B. OXYGEN-ACTIVATED SLUDGE PLANTS UNDER CONSTRUCITON AS OF JUNE 1976

Location	Design Flow (mgd)*	Installed O_2 Supply Capacity (tons/day)†	Application‡	O_2 Dissolution System§	O_2 Supply System∝
USA					
1. Appleton Papers Div. of N.C.F. - Appleton, Wisc.	6.5	14	I-PP	UNOX (A)	PSA
2. Baltimore, Md.	70	75	M	UNOX (A)	CRYO
3. Baton Rouge, La.	16	12	M	UNOX (A)	PSA
4. Broken Arrow, Okla.	4	3.5	M(d)	UNOX (A)	LIQ
5. Cedar Rapids, Iowa	33	120	M	UNOX (A)	CRYO
6. Chicopee, Mass.	15.5	17	M	UNOX (A)	PSA
7. Cincinnati, Ohio	1.2	50	M(Z)	UNOX (A)	CRYO
8. Crown Zellerbach Corp. - Antioch, Calif.	5.5	10	I-PP	UNOX (A)	PSA
9. Dade County (North), Fla.	60	100	M	OASES (A)	CRYO
10. Danville, Va.	24	33	M	UNOX (A)	PSA
11. Denver (#1), Colo.	72	80	M	UNOX (A)	CRYO
12. Detroit (#2), Mich.	600	450	M	OASES(T&A)	CRYO
13. Dow Chemical Co. - Plaquemine, La.	12.7	69	I-C	UNOX (A)	PIPE
14. Dubuque, Iowa	16	26	M	OASES (A)	CRYO
15. Duluth, Minn.	43.6	80	M	UNOX (A)	CRYO
16. East Bay Municipal Utility District (#1) - Oakland, Calif.	120	250	M	UNOX (T)	CRYO
17. East Bay Municipal Utility District (#2) - Oakland, Calif.	1.5	1.5	M	MAROX (R)	PIPE
18. Euclid, Ohio	22	28	M	UNOX (A)	PSA
19. Exon Chemical Co. - Baton Rouge, La.	9	35	I-PC	UNOX (A)	CRYO
20. Fairbanks, Alaska	8	13	M(d)	UNOX (A)	PSA
21. Fond du Lac, Wisc.	11	26	M	UNOX (A)	PSA
22. Ft. Lauderdale, Fla.	22	55	M(n)	OASES (A)	CRYO
23. Harrisburg, Pa.	35.4	50	M	UNOX (A)	CRYO
24. Hillsboro, Ore.	15	9	M	UNOX (A)	PSA
25. Hopewell, Va.	57.63	100	M	UNOX (A)	CRYO
26. Hot Springs, Ark.	12.1	11.5	M	UNOX (A)	PSA
27. Jacksonville (#2), Fla.	5	16	M	UNOX (A)	PSA
28. Kittanning, Pa.	1.5	1	M	UNOX (A)	PIPE
29. Lewisville, Tex.	6	7	M	UNOX (A)	PSA
30. Littleton/Englewood, Colo.	20	21	M	MAROX (R)	CRYO
31. Longview Fiber - Longview, Wash.	30	40	I-PP	UNOX (A)	CRYO
32. Louisville, Ky.	105	100	M	UNOX (T)	CRYO
33. Loxahatchee, Fla.	4	9	M(n)	UNOX (A)	PSA
34. Mahoning County, Ohio	4	7	M(n)	UNOX (A)	PSA
35. Miami, Fla	55	80	M	UNOX (A)	CRYO
36. Middlesex, N.J.	120	450	M(d)	UNOX (T)	CRYO
37. Minneapolis, Minn.	1	1	M	MAROX (F)	LIQ
38. Mobile, Ala	28	26	M	UNOX (A)	PSA
39. Mosinee Paper Corp. - Mosinee, Wisc.	6	13	I-PP	UNOX (A)	PSA
40. Muscacine, Iowa	13	80	M	UNOX (A)	CRYO
41. Nekoosa Papers, Inc. - Port Edwards, Wisc.	35	52.8	I-PP(b)	UNOX (A)	CRYO
42. New Orleans, La.	122	140	M	OASES (A)	CRYO
43. North San Mateo, Calif.	8	10	M	UNOX (A)	PSA
44. Pensacola, Fla.	24	40	M(n&O)	OASES (A)	CRYO
45. Philadelphia (Southwest), Pa.	210	90	M	UNOX (A)	CRYO
46. Pima County, Ariz.	25	22	M	UNOX (A)	PSA

APPENDIX B, Continued

Location	Design Flow (mgd)*	Installed O_2 Supply Capacity (tons/day)†	Application‡	O_2 Dissolution System§	O_2 Supply System∝
47. St. Regis Paper Co. - Tacoma, Wash.	34	40	I-PP	UNOX (A)	CRYO
48. Salem, Ore.	26.5	36	M	UNOX (A)	PSA
49. Shell Oil Co. - Norco, La.	4.3	50	I-PC	UNOX (A)	CRYO
50. Springfield, Mo.	30	36	M(O_3)	UNOX (A)	PSA
51. Sunkist Growers, Lemon Products Div. - Corona, Calif.	1.75	50	I-FP	UNOX (A)	CRYO
52. Tahoe/Truckee, Calif.	8	4	M	UNOX (A)	PSA
53. Tampa, Fla.	51	120	M(n)	UNOX (A)	CRYO
54. Tauton, Mass.	8.4	20	M(n)	UNOX (A)	CRYO
55. Thilmany Pulp & Paper Co. - Kaukauna, Wisc	22	10	I-PP	UNOX (A)	PSA
56. Tonawanda, N.Y.	30	32	M	UNOX (A)	CRYO
57. Two Bridges, N.J.	7.5	6	M	UNOX (A)	PSA
TOTAL	2339.58	3328.3			
Mexico					
1. Fundidora Steel Co. - Monterrey	13.7	12.7	I-S	UNOX (A)	PIPE
Europe					
1. ARA BIRS II, Switzerland	18	20	M	UNOX (A)	PSA
2. Bayer-Elberfeld - Dusseldorf, Germany	1.8	50	I-C	UNOX (A)	CRYO
3. Copenhagen, Denmark	110	160	M	UNOX (A)	CRYO
4. Palmersford, England	1.2	2	M(n)	UNOX (A)	PSA
5. Union Carbide Belgium - Antwerp, Belgium	0.71	16	I-C	UNOX (A)	PSA
TOTAL	131.71	248			
Japan					
1. Mitsubishi Chemical Industries - Kitakyushu City	3.09	n.d.**	I-DS	UNOX (A)	n.d.**
2. Mitsui Toatsu Chemicals Takaishi City	1.71	n.d.	I-PC	UNOX (A)	n.d.
3. Tokiwa Sangyo - Owari Asahi City	3.7	n.d.	I-PP	UNOX (A)	n.d.
TOTAL	8.50				

---

*1 mgd = 0.044 cu m/sec

†1 ton/day = 0.907 metric ton/day

‡I-C = Industrial-Chemicals
I-DS = Industrial-Dyestuffs
I-FP = Industrial-Food Processing
I-PC = Industrial-Petrochemical
I-PP = Industrial-Pulp & Paper
I-S = Industrial-Steel
M = Municipal
(b) = Conventional O_2 treatment plus black liquor oxidation
(d) = Conventional O_2 treatment plus aerobic sludge digestion
(n) = Conventional O_2 treatment plus nitrification
(O_3) = Conventional O_2 treatment plus effluent ozonation
(Z) = Treatment of Zimpro supernatant only

§MAROX = FMC Corp.
OASES = Air Products & Chemicals, Inc.
UNOX = Union Carbide Corp.
(A) = Surface aerators (with or without bottom mixers)
(F) = Fixed active diffusers
(R) = Rotating active diffusers
(T) = Submerged turbines

∝CRYO = On-site cryogenic oxygen gas generation
LIQ = On-site liquid oxygen storage and vaporization
PIPE = Pipeline transport of oxygen gas from a nearby off-site oxygen generating facility
PSA = On-site pressure swing adsorption oxygen gas generation

**n.d. = no data

APPENDIX C. OXYGEN-ACTIVATED SLUDGE PLANTS BEING DESIGNED AS OF JUNE 1976

Location	Design Flow (mgd)*	Design O_2 Supply Capacity (tons/day)†	Application‡
USA			
1. Amherst, N.Y.	24	40	M
2. Augusta, Me.	8	9	M
3. Baldwinsville, N.Y.	9	19	M
4. Clay, N.Y.	10	10	M
5. Clinton, N.C.	3	9	M
6. Concord, N.C.	25	80	M
7. Dade County (South), Fla.	40	130	M
8. Easton, Pa.	10	14	M
9. Greenville, S. C.	5	8	M
10. Hamburg (South Towns), N.Y.	12	14	M
11. Hampton Roads Sanitary District, Va.			
(a) Army Base	19	21	M
(b) Atlantic	36	40	M
(c) Boat Harbor	26	30	M
(d) Lamberts Point	37	42	M
12. Hannibal, Mo.	4.25	9	M
13. Holyoke, Mass.	22	22	M
14. Houston, Tex.	200	305	M
15. Indianapolis (Belmont), Ind.	125	180	M
16. Indianapolis (Southport), Ind.	125	180	M
17. Kansas City, Kan.	54	80	M
18. Kaukauna, Wisc.	6.1	13	M
19. Lebanon, Pa.	8	24	M
20. Los Angeles (Hyperion), Calif.	330	340	M
21. Los Angeles County (JWPCP), Calif.	500	500	M
22. Maryland City, Md.	4	7	M
23. Montgomery County, Pa.	10	14	M
24. Monticello, N.Y.	6	6	M
25. Murfreesboro, Tenn.	8	13	M
26. New Rochelle, N.Y.	14	16	M
27. Orlando, Fla.	24	50	M
28. Passaic Valley, N.J.	300	1000	M
29. Philadelphia (Northeast), Pa.	150	100	M
30. Philadelphia (Southeast), Pa.	100	80	M
31. Red Springs, N.C.	1.5	5	M
32. Sacramento, Calif.	150	200	M
33. San Francisco, Calif.	180	100	M
34. South Cobb County, Ga.	24	40	M
35. Sussex County, Del.	8	11	M
36. Tri-Municipal Sanitary District - Poughkeepsie, N.Y.	14	9	M
37. York, Pa.	8	13	M
38. Texas City, Tex.	7.5	11	M
TOTAL	2647.35	3794	

---

* 1 mgd = 0.044 cu m/sec

† 1 ton/day = 0.907 metric ton/day

‡ M = Municipal

CURRENT RESEARCH RELATED TO HEAT CONDITIONING OF WASTEWATER SLUDGE

B. V. Salotto, J. B. Farrell, J. E. Smith, Jr., E. Grossman, III
Municipal Environmental Research Laboratory
Cincinnati, Ohio 45268

ABSTRACT

Low and high temperature heat treatment of sludge produce liquors which are easily dewatered but create other problems. High temperature - high pressure processes are costly, energy intensive, and less safe to operate than are low temperature processes. However, low-temperature processes produce liquors which need further treatment and they add to the overall cost of the heat treatment process. Current research is aimed at finding economical, energy-saving, processes to treat and recycle heat treatment liquor. It is also aimed at developing heat treatment processes which are less energy intensive but attain the same degree of oxidation as occurs in a high temperature heat treatment process. In the first case laboratory conventional anaerobic treatment of the liquor has been found to be a satisfactory method of treatment under low loading conditions; however, a limiting factor is the liquid throughput rate. Data from operation of 3 lab digesters indicate that at solids retention times of less than 10 days, some provisions must be made for return of biomass to the system, otherwise washout may occur. Data are also presented which would seem to indicate a new and novel approach to low temperature heat treatment of sludge. It is known as the Puretec WETOX process. Pilot plant results indicate the achievement of high oxidation rates equivalent to high temperature-high pressure, heat treatment systems. Other advantages are claimed such as potential for heat and byproduct recoveries.

INTRODUCTION

One of the most controversial processes being employed in the wastewater treatment industry today is that of thermal sludge conditioning. The application of heat under pressure (300 to 500°F and 150-400 psig) for protracted periods conditions the sludge by causing profound changes in its nature and composition. Sewage sludges are essentially cellular material containing intracellular gel and extracellular zoogleal slime of equal amounts of carbohydrate and protein. The action of heat is to break open the cells and release the mainly proteinaceous protoplasm. The heat also breaks down the protein and zoogleal slime, producing a dark brown liquor consisting of soluble polypeptides, ammonia nitrogen, volatile acids and carbohydrates (1). Generally, the end results of this conditioning and "pressure cooking" step are a heat treated sludge liquor which requires some sort of handling and a residue of considerably increased dewaterability. The magnitude of these two effects is the subject of a large amount of debate. The purveyors of heat conditioning equipment claim that after conditioning virtually no chemical conditioning of the sludge is required for dewatering, that it dewaters at a very high rate, and that the cake solids produced are sufficient to permit autogenous incineration with recovery of sufficient waste heat to drive the heat conditioning process. Further they claim little difficulty in handling the heat treated sludge liquor. The critics of heat treatment disagree with all these claims and note the following reported analyses of cooking liquor: (2)

BOD_5 = 5000 to 15000 mg/l
COD = 10000 to 30000 mg/l
Ammonia = 500 to 700 mg/l
Phosphorus = 150 to 200 mg/l

About 20 to 30% of the liquor's COD has been found not biodegradable in a thirty-day period. While the volume of cooking liquor from an activated sludge plant will amount to 0.75 to 1.0% of the wastewater flow; on a BOD and solids loading basis, the liquor can represent from 30 to 50% of the normal loading on the aeration system from settled sewage. The critics also point to the mechanical and odor problems that have occurred at plants like Colorado Springs, Colorado; Chattanooga, Tennessee; and Speedway, Indiana. Then some plants like Kalamazoo, Michigan, have found that the costs of heat treatment have rapidly escalated and treatment schemes had to be installed to handle the processes' liquor.

It is possible by operating at even higher temperatures and pressures in the presence of oxygen to accomplish a high level of organic destruction by the process of wet air oxidation. Operating conditions range from 450 to 600°F and from 800 to 1800 psig depending upon the degree of oxidation required. This process normally does not require external heating for sustaining combustion once it is started. Equipment includes grinders, high pressure pumps, heat exchangers, high pressure reactors and separators. The end products of wet air oxidation processes are a carbonaceous ash and sludge liquor. Criticisms of this process are similar to those of the low pressure and low temperature system but include safety along with serious operating and maintenance difficulties. The pollutant concentration of the resultant cooking liquor stream varies according to the degree of treatment and the particular sludge involved. Typical concentrations are: (3)

COD = 10000 to 20000 mg/l
BOD $\leq$ 14000 mg/l
Ammonia = 500 to 1000 mg/l
Solids $\leq$ 2.5%
PH = 4 to 4.8

Because of the controversy surrounding the two thermal sludge processes, EPA has been conducting research to further characterize heat treated sludge liquor and evaluate methods for treating it. This paper will report on these efforts. Further the paper will report on a new wet air oxidation process, which seeks to eliminate some of the disadvantages of the older system. This new process may also facilitate the recovery of metals from sludge. As such the types and quantities of metals in sludge will be discussed along with the feasibility of their recovery.

LIQUOR FROM LOW TEMPERATURE (< 400°F) HEAT TREATMENT

Anaerobic Treatment

Probably the major disadvantage of heat treatment of sludge is the concentrated supernatant that is produced. If the BOD discharged from a wastewater treatment plant is not to be increased, more treatment capacity is needed. For the activated sludge process, this means a larger aeration basin, greater air demand, and more biological sludge to be processed. An alternative approach is to treat the sludge anaerobically. Anaerobic treatment reduces BOD with a much smaller increase in biomass than occurs with aerobic treatment, no power cost for air compression is incurred, and usable fuel gas is produced. Treatment can be carried out by using conventional anaerobic digestion or an anaerobic filter (4). Both of these methods have advantages. Since most plants that are using heat treatment previously used digestion, digesters are generally available for use. The anaerobic filter is probably the better choice if new construction is required. Research has been commenced in our laboratory on both of these approaches, but only the anaerobic digestion approach is far enough along for presentation of preliminary results.

In order to measure such things as gas production and COD destruction under different loading conditions, three small laboratory anaerobic digesters were set up, each containing 3.5 liters of digesting material in a 4 liter digesting bottle. Figure 1 shows a schematic picture of one of the digesters. Provision was made to maintain temperature constant, to stir the contents of the digester with a magnetic stirrer, and to collect and measure each day's production of gas. Not shown in the figure is a gas analyzer apparatus to measure CO_2 content of the gas mixture.

Data were collected so as to be able to determine or calculate such items as loading rates, COD, BOD, volatile solids destruction and gas yield.

At first all three digesters were fed 150 ml of heat treatment liquor each day for 12 weeks. Thus, all three digesters were operated at the same loading rate. A supply of heat treatment liquor was obtained from a local sewage treatment plant which heat treated its sludge by the low pressure Zimpro process. Each day 150 ml of digesting liquor was removed from each digester and analyzed for COD, BOD, total and volatile solids, pH, and

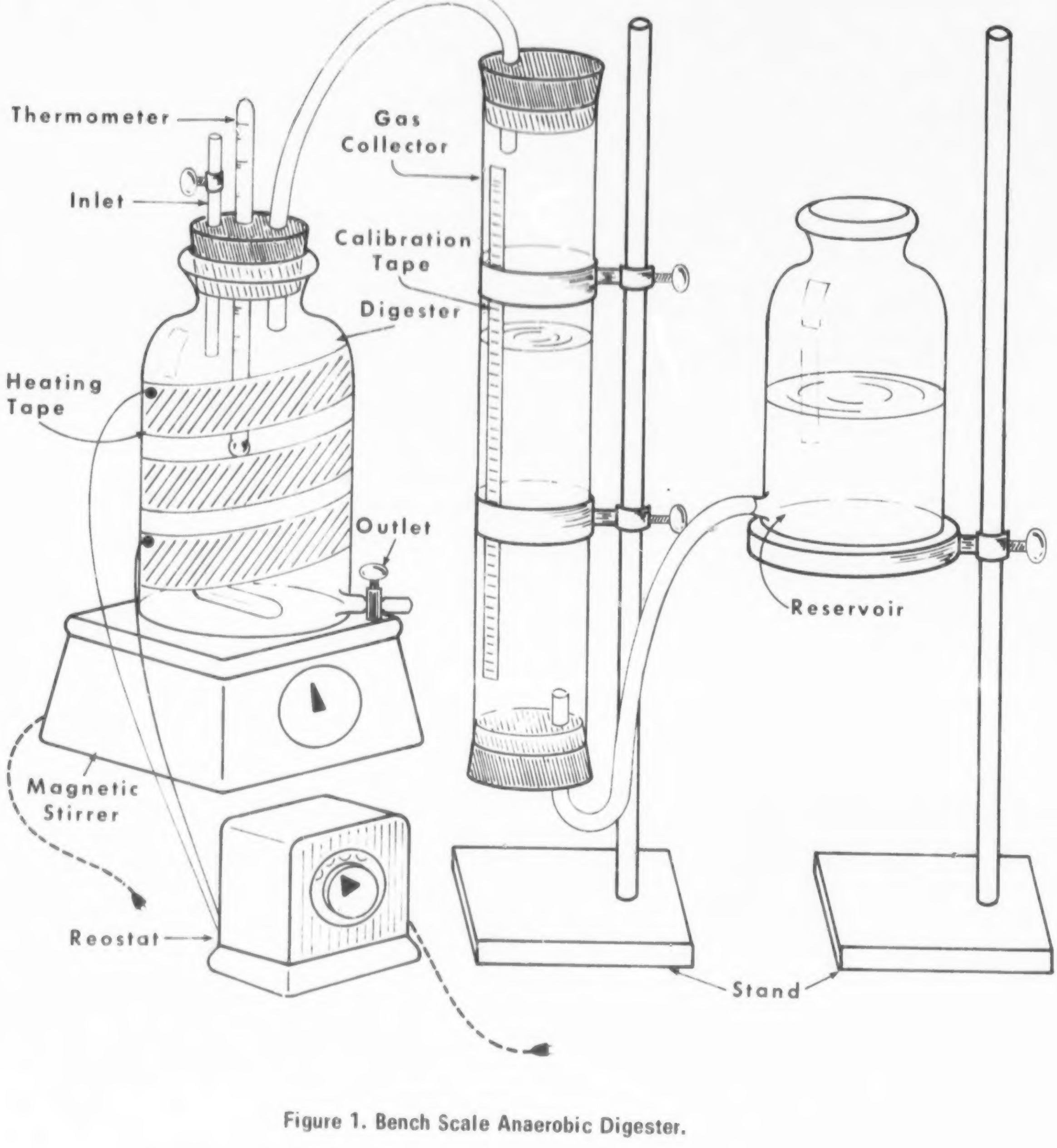

Thermometer
Inlet
Gas Collector
Calibration Tape
Digester
Heating Tape
Outlet
Reservoir
Magnetic Stirrer
Reostat
Stand

Figure 1. Bench Scale Anaerobic Digester.

the like. The hydraulic detention time for all three digesters was 24 days.

The manner of withdrawing digesting liquor is of interest. In all cases, the digester was thoroughly mixed just before the withdrawal was made. This procedure is equivalent to the withdrawal method used for conventional sludge digestion. A portion of the active biomass is withdrawn with the digesting liquor. This procedure is suitable provided the residence time in the digester is high enough so that "washout" of the biomass does not occur. It may be possible to push anaerobic digestion of supernatant to very high rates if biomass is returned to the digester, or is not removed by allowing settling before withdrawal. This latter procedure will be tested in a later phase of the investigation.

Some preliminary results of the digester study have been obtained. Table 1 lists average characteristics of the heat treatment liquor feed. It should be pointed out that the more settleable solids were allowed to settle and the supernatant stored before feeding the digesters. This was done so that the liquor might more nearly resemble the filtrate derived from vacuum filtration of a heat treated sludge.

TABLE 1

AVERAGE CHARACTERISTICS OF HEAT TREATMENT LIQUOR FEED (TO ANAEROBIC DIGESTERS)

PARAMETER	UNIT	AMOUNT
pH	pH Unit	5.6
ALKALINITY	mg/l $CaCO_3$	640
TOTAL SOLIDS	%	0.55
VOLATILE SOLIDS	%	79
COD	mg/l	6200
BOD	mg/l	2440
TKN	mg/l	590
NH_3	mg/l	240
VOLATILE ACIDS	mg/l ACETIC	1340
PHOSPHORUS	mg/l	40

As the data indicates, the percent total solids is low (0.55%). Note that the pH is low as is the alkalinity. This was of some concern because in normal digestion these values are higher. As might be expected, COD and BOD are high although heat treatment liquors are often much higher. The nitrogen and phosphorus values are shown to indicate that these essential nutrients are present in sufficient amounts to insure good digestion.

Data for all 3 digesters operating at the same loading rate were so similar that the tabulation shown in Table 2 is typical of all 3 digesters. This simply indicates that very little variation occurred. The excellent agreement of results among the three digesters gives confidence that differences in performance noted in later tests when operating parameters for the digesters are different will be the effect of the changed conditions and not lack of reproducibility within the units.

TABLE 2

DIGESTER OPERATION: OPERATING PARAMETERS AND ANALYSES OF DIGESTED LIQUOR*

OPERATOR	UNIT	AMOUNT
FEED RATE	ml/day	150
pH	pH Unit	7.0
ALKALINITY	mg/l $CaCO_3$	1910
VOLATILE ACIDS	mg/l ACETIC	63
TEMPERATURE	°C	35±0.2
TOTAL SOLIDS	%	0.35
VOLATILE SOLIDS	%	54.5
COD/BOD	mg/l	2160/220
DETENTION TIME	DAYS	24
LOADING RATE, VS	Kg/m^3/day	0.174
VOLATILE SOLIDS REDUCTION	%	59
GAS YIELD, PER		
VOLATILE SOLIDS DESTROYED	m^3gas/Kg VS DESTROYED	0.95
VOLATILE SOLIDS ADDED	m^3gas/Kg VS ADDED	0.62

*Average results of three digesters

A comparison of the analyses of the feed (Table 1) and the product (Table 2) shows that pH increased to a normal value of 7.0, alkalinity increased to 1900 mg/l, and volatile acids dropped to a low value. These data show a good operating digester. Variations of the properties of the digested liquor with time are presented in Figure 2. As can be seen, pH was quite stable around 7, whereas alkalinity and volatile acids fluctuated somewhat during the 12 week period.

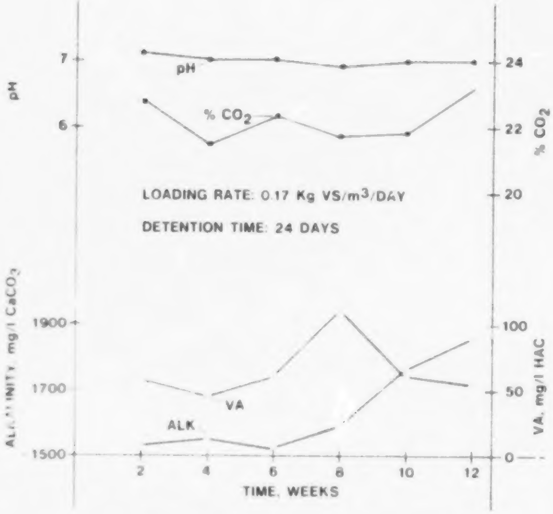


Figure 2. Characteristics of Digested Heat Treatment Liquor- Unit 1.

During the last 12 week phase of the study, each digester was operated at different loading rates in which 300 ml, 450 ml, and 600 ml of heat treatment liquor was fed daily to the 3 digesters. Unfortunately, an accident occurred with the digester being fed 600 ml of liquor per day so that no data can be presented for that digester. Table 3 shows results of digester operation at the 300 and 450 ml feed rate after a run of 12 weeks compared with operation of 150 ml for the other 12 week period. Note that efficiency of the anaerobic process decreases as the loading rate increases. Less COD, BOD, or volatile solids are being destroyed as the loading rate increases. Note also that gas yield, % CO_2, and volatile acids change with changing loading rate. Surprisingly, no adjustment of pH or addition of alkalinity was needed for digester operation. Alkalinity built up to approximately 3000 mg/l at the higher rates.

These studies will continue at higher loading rates to determine at what point digester failure occurs. Comparisons will be made against operation in which biomass is retained in the digester, to determine the increase in throughput rate that can be achieved with this mode of operation.

TABLE 3

DIGESTER OPERATION AT DIFFERENT LOADING RATE
(DIGESTED SUPERNATANT)

	LOADING RATES, ml/DAY		
OPERATOR	150	300	450
pH	7.0	7.2	7.1
ALKALINITY, mg/l	1910	3310	3130
TOTAL SOLIDS, %	0.35	0.40	0.44
VOLATILE SOLIDS, %	54	61	64
COD, mg/l	2160	3600	4510
BOD, mg/l	222	660	1040
DETENTION TIME, DAYS	24	12	8
VOLATILE SOLIDS LOADING, Kg/m^3/day	0.174	0.657	0.985
COD/BOD REDUCTION, %	85/91	68/87	60/80
GAS YIELD			
m^3gas/Kg VS DESTROYED	0.95	0.82	0.61
m^3gas/Kg VS ADDED	0.62	0.52	0.59
% CO_2 GAS MIXTURE	22	28	29
VOLATILE ACIDS, mg/l	65	114	410

Aerobic and Physical-Chemical Treatment

Studies in Great Britain have indicated that heat treatment liquor can be reduced in BOD by the activated sludge process and by trickling filters. Corrie, in a relatively recent publication (2), reports considerable success in which treatment by the activated sludge process is followed by adsorption on activated carbon. By this procedure both BOD and COD are reduced. Typically, biological processes are not very effective in reducing the COD of heat treatment liquors. Corrie's procedure will be evaluated at Lake County, Ohio, where a heat treatment unit has been installed under an EPA grant (5). At Lake County, the effectiveness of high lime treatment of heat treatment liquor will also be evaluated. Unfortunately, these studies have not been commenced as yet because of unusual difficulties encountered in starting up the heat treatment unit.

A NEW APPROACH TO HIGH TEMPERATURE OXIDATIVE SLUDGE PROCESSING

Puretec Wet Oxidation Process

Oxidation of water-based sludges at high temperatures in the presence of air was pioneered by the Zimpro Division of Sterling Drug. Although several installations were made (e.g., Chicago, Wheeling, Akron), Zimpro has had its greatest commercial success with low oxidation units (<400°F). The primary disadvantage of the high temperature units has been the high temperature (>500°F) and high pressure (ca. 1600 psi) conditions of operation.

In the Zimpro process, air and sludge are admixed and pumped through a vertical unmixed reactor. Numerous studies have indicated (6) that agitation improves reaction rate of multiphase processes. However, mechanical problems for stirring high pressure reactors are formidable. The Resource Recovery Systems Division of the Barber-Colman Company has developed a processing system, the Puretec Process, and equipment that seems to overcome these difficulties. Operating temperatures are about 450°F and pressures about 600 psi. The U.S. EPA is financing a grant to the City of Philadelphia to investigate the Puretec Process at their Northeast Wastewater Treatment Plant on a sufficiently large scale to accurately determine process costs, claimed heat recovery benefits, and achievable sludge destruction. (7)

The Philadelphia Puretec Installation

The Puretec WETOX unit, Model 6-54, can treat a sludge flow of 3000 gallons per hour or 16 tons of dry solids per day. (8) In the process, oxidation of liquid waste is carried out continuously at 430°F and 600 psig in an acidic medium (3 g/l H_2SO_4). The oxidation proceeds in a horizontal autoclave containing 6 compartments with individual stirring in each compartment. A process flow diagram is shown in Figure 3. Macerated sludge is pumped through a liquid and vapor phase tube-in-tube heat exchanger and into the front end of the reactor. The sludge is mixed with compressed air, and reacted in the first chamber from which it overflows successively into the several following compartments undergoing nearly complete oxidation. Acid addition as well as stirring enables a high rate of oxygen transfer to the sludge causing a rapid destruction of organic matter. Sulfur compounds are oxidized to sulfates. In the final compartment liquid and vapor phases are separated and conducted separately to heat exchangers for thermal energy exchange with incoming sludge. After cooling the vapor phase and the liquid phase are let down to atmospheric pressure. At this point a variety of post-treatment steps may be used to purify the effluent side-streams such as the lime treatment step shown in Figure 3.

Pilot Plant Results at Irvine, California

Some impressive results have been achieved with the 4-10 model, 20 gallon/hr pilot plant Puretec unit at Irvine, California, the location of the Barber-Colman Company Research Lab. Figure 4 shows good improvement in COD reduction versus speed of agitation. Note that at high rpm after a reaction time of 40 minutes one obtains an 80 percent reduction in COD. Figure 5 shows the increase in COD destruction as a function of acid

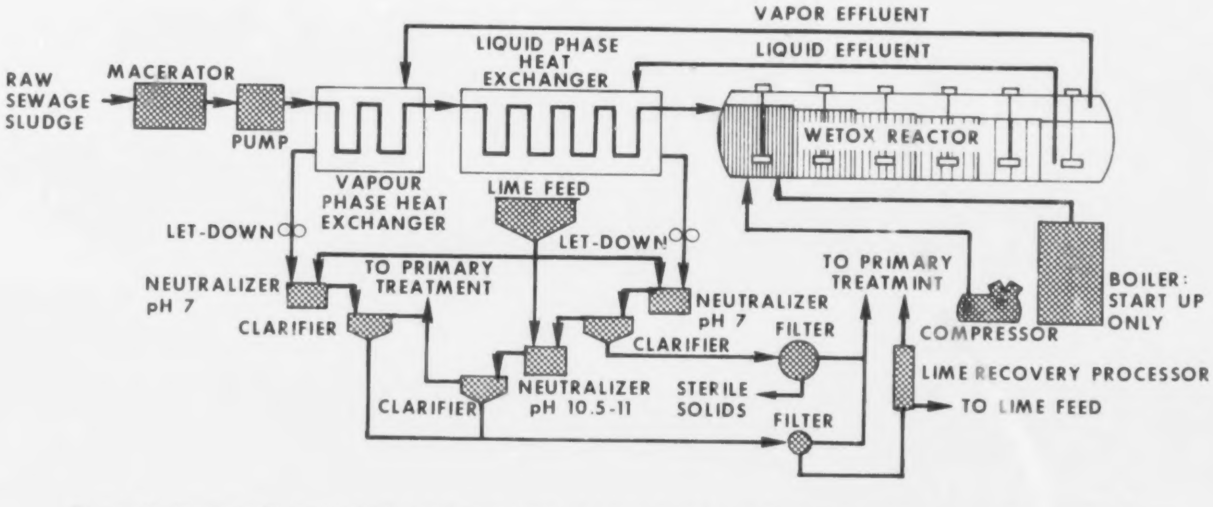

VAPOR EFFLUENT
LIQUID PHASE HEAT EXCHANGER
LIQUID EFFLUENT
RAW SEWAGE SLUDGE
MACERATOR
PUMP
WETOX REACTOR
VAPOUR PHASE HEAT EXCHANGER
LIME FEED
LET-DOWN
LET-DOWN
NEUTRALIZER pH 7
TO PRIMARY TREATMENT
TO PRIMARY TREATMINT
BOILER: START UP ONLY
NEUTRALIZER pH 7
CLARIFIER
COMPRESSOR
FILTER
CLARIFIER
LIME RECOVERY PROCESSOR
NEUTRALIZER pH 10.5-11
STERILE SOLIDS
CLARIFIER
TO LIME FEED
FILTER

Figure 3. Process Flowsheet of the Wetox Unit. A Lime Treatment System is added on to Further Purify the Wetox Effluents. The Combined System (Wetox Plus Lime Treatment) is Registered Under the Trade Name of Puretec System.

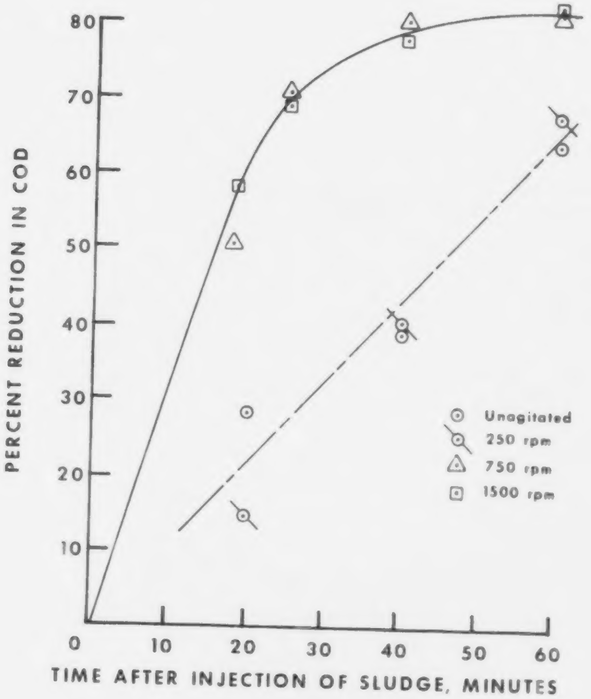


Figure 4. Effect of Agitation in Wetox.

addition. The effect of the acid is to shorten the reaction time. The usual dose of H_2SO_4 is 3 g/l; higher doses, as shown in Figure 5, do not materially affect the reaction time.

Advantages Claimed for the Puretec Process (8)

No scale or corrosion problems - corrosion is inhibited in the process by using titanium in piping and valves and by lining the inside of the reactor with lead and acid-resistant brick. The chief advantage here is that corrosion due to Cl^- or $SO_4^{=2}$ ions is prevented. Acid also prevents scaling in the reactor or in piping.

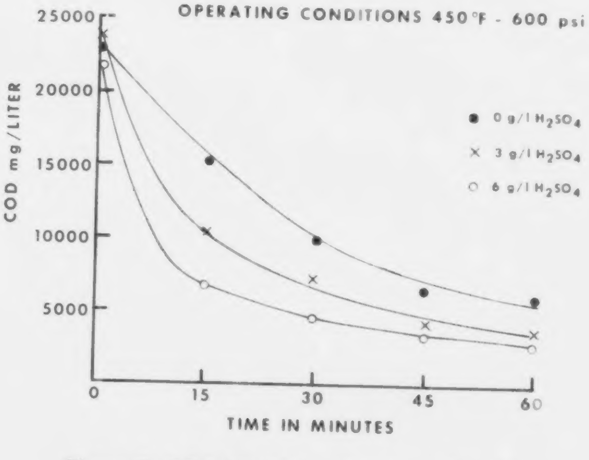


Figure 5. COD Reduction of Orange County Sanitation District Primary Sludge.

No odor or noise problems - the use of adequately dispersed oxygen in an acidified medium results in a rapid oxidation of sulfur and nitrogen-bearing compounds to sulfate and ammonia. The reaction does not form the intermediate organic compounds which tend to be malodorous in conventional processes. Raw primary or unstabilized sludge is ordinarily extremely offensive; however, under the Puretec oxidizing conditions very little odor is present. Carefully muffled low speed compressors, which supply the high pressure air, specially designed high pressure sludge pumps, and the design of the let down valves all combine to minimize the noise level usually associated with operation of heat treatment units.

The Puretec WETOX process is competitive - analysis of costs of the process indicate a figure of $45-50 per ton of dry solids; this cost has not been adjusted by credits as a result of by-product recoveries. Adjustment downward as a result of credits could materially lower the overall cost of sludge treatment and disposal.

FEASIBILITY OF METALS RECOVERY

Concentration of Metals in Sludge

Before recovery of metals can be discussed, it is important to establish concentration levels in order to determine whether recovery is feasible. It is well to note that quantities to be processed and concentrations can be well below the normal levels considered to be practical for metal refining operations, because there is a credit to be taken for reduction of potential hazard and disposal costs of the residue.

Fortunately, considerable information has been collected on the concentrations of metals in sludge. The Ultimate Disposal Section of the Municipal Environmental Research Laboratory's Wastewater Research Division in Cincinnati has had an ongoing program in which sludges from various treatment plants around the United States are analyzed. Results have been published by Salotto et al (9). Average concentrations of 13 metals are shown in Table 4. It can be noted in Table 4 that the geometric mean is closer to the median value than the arithmetic mean. This indicates that a logarithmic mean distribution is a good estimate of the type of distribution of metal concentrations. This in fact is the case for all metals examined, and is illustrated by data for zinc in Figure 6. The implication of a logarithmic mean distribution, when coupled with a large logarithmic standard deviation or "spread", is that there will be a fairly large number of plants with concentrations substantially in excess of the median. This indicates that the median concentration is probably too low a figure to use in estimating the practicality of recovery, but a number substantially higher should be used.

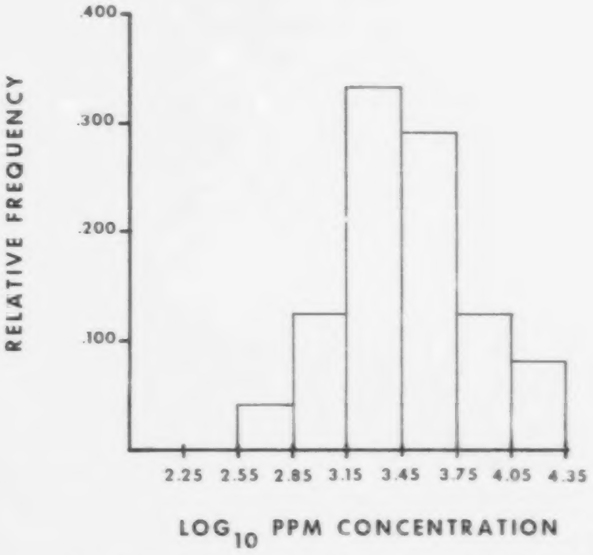


Figure 6. Histogram of Zinc in Digested Sludge.

Metal Recovery by the Puretec WETOX Process

As mentioned earlier, because of the use of sulfuric acid in the Puretec WETOX process, the potential for metal recovery looks quite good. Dr. W. Martin Fassell, vice-president and developer of the process, in his article "Sewage Sludge at a Profit" (10), has reported on tests by Barber-Colman on recovery of metals from ash after wet-oxidation. The effect on the original level of acid addition on the percent of the metal remaining in sludge treated by the Puretec process is shown in Table 5. It is interesting to observe from these data that increasing amounts of acid did not necessarily dissolve more of the metals, in fact, very little lead, silver and titanium were dissolved out of the ash. Evidently the ion products of insoluble forms of these elements were sufficiently low that dissolution was not possible. On the other hand, copper, cadmium, and zinc are quite soluble, and afterwards can easily be recovered by precipitation as sulfides according to the claim of Dr. Fassell (10). At any rate, it would seem, because of the presence of an acid side effluent, that the potential for metal recovery is quite good.

TABLE 4 (See Reference 9)

AVERAGE CONCENTRATIONS OF METALS IN DIGESTED SLUDGE

(ALL FIGURES MG/KG* DRY SLUDGE BASIS)

METAL	ARITHMETIC MEAN	ARITHMETIC STD.DEV. + -	GEOMETRIC MEAN	GEOMETRIC STD.DEV. + X	MEDIAN 50% VALUE
SILVER	250	230	190	1.99	100
BORON	430	310	380	1.58	350
CADMIUM	75	104	43	2.47	31
CALCIUM	36,500	23,800	31,100	1.77	30,000
CHROMIUM	1,660	1,920	1,050	3.22	1,100
COBALT	350	220	290	1.88	< 100
COPPER	1,590	1,670	1,270	1.95	1,230
MERCURY	10	18	6.5	2.34	6.6
MANGANESE	1,300	2,290	475	3.67	380
NICKEL	680	620	530	1.88	410
LEAD	2,750	2,350	2,210	1.82	830
STRONTIUM	520	670	290	2.70	175
ZINC	4,210	3,500	2,900	2.40	2,780

* A MG/KG - PPM.

< LESS THAN

TABLE 5 (See Reference 10)

PERCENT OF METALS PRESENT IN WET OXIDATION ASH VERSUS ACID ADDITION

METALS WT.%	H_2SO_4 ADDITION TO SLUDGE FEED GRAMS PER LITER			
	0	6	12	18
COPPER	93	66	53	7
LEAD	100	90	77	77
ZINC	100	66	44	0
CADMIUM	97	50	38	0
SILVER	98	94	87	88
IRON	98	91	92	65
TITANIUM	100	100	100	100
pH	5.0	3.5	1.8	1.1

SUMMARY

Low pressure heat treatment, a sludge-conditioning process that has seen almost explosive adoption by wastewater treatment plants in the United States, solves the sludge dewatering problem in most cases, but creates nearly as many problems as it solves. One of its serious drawbacks is the creation of a concentrated recycle liquor. It is anticipated that there will frequently be a need to separately treat this concentrated liquor to reduce the load on the treatment plant. Aerobic and physical-chemical means have been demonstrated to be effective for separate treatment but they are costly and require manpower to operate. Anaerobic processes are worth considering as an alternative because they are typically less labor and power intensive. The present investigation demonstrates that conventional digestion satisfactorily reduces BOD of heat treatment liquor. Continuing studies will be directed towards achieving higher treatment rates by modified anaerobic digestion by recycling biomass, and by the anaerobic filter.

High temperature oxidative processes for sludge disposal have been poorly accepted because of cost and extreme processing temperatures and pressures. The Puretec process claims to avoid these extremes by use of a stirred reactor, for which high contact efficiency is claimed. A demonstration at the City of Philadelphia, sufficiently large to demonstrate economics of performance, will be carried out with EPA support. The unique configuration of the reactor and the use of acidic conditions offers the possibility of recovery of heat, byproduct streams such as ammonia and acetic acid and some metals.

ACKNOWLEDGEMENT

The authors wish to acknowledge Mrs. Patricia Tutt, Physical Science Aid, for her very valuable assistance in the anaerobic digester study of heat treatment liquor. We also wish to acknowledge the help of Dr. W. Martin Fassell, Vice-President, Barber-Colman Company, who gave us permission to use pilot plant data and loaned us slides for the presentation.

References

1. Brooks, R.B., "Heat Treatment of Sewage Sludge," Water Pollution Control (G.B.) 69, 221 (1970).

2. Corrie, K.D., and Wycombe, R.D.C., "Use of Activated Carbon in the Treatment of Heat Treatment Plant Liquor," Water Pollution Control (G.B.) 71, 629 (1972).

3. Fischer, W.J., and Swanwick, J.D., "High Temperature Treatment of Sewage Sludges," Water Pollution Control (G.B.) 70, 355 (1971).

4. McCarty, P.L., "Anaerobic Treatment of Soluble Wastes" in Advances in Water Quality Improvement, edited by Glazer and Eckenfelder, University of Texas Press, Austin (1968).

5. EPA Grant 11010 DKI, "Porteous Process for Heat Treatment of Sludge," Grant awarded to Lake County, Ohio, Sept. 1971. (Project officer, B.V. Salotto, EPA, Municipal Environmental Research Laboratory, Cincinnati, Ohio 45268).

6. Farrell, J.B., and Haas, P.A., "Oxidation of Nuclear Grade Graphite by Nitric Acid and Oxygen," IECC Process Design and Development 6, 277 (July 1967).

7. EPA Grant S-803644-01-1, "Puretec Wet-Oxidation of Municipal Sludge," Grant awarded to the Philadelphia Water Department, Philadelphia, Pennsylvania, May 30, 1975. (Project officer, B.V. Salotto, EPA, Municipal Environmental Research Laboratory, Cincinnati, Ohio 45268).

8. Seto, P., "Evaluation of the Barber-Colman WETOX Process for Sewage Sludge Disposal," Training and Technology Transfer Division (Water), EPS, Ottawa, Canada, KIA OH3, Project No. 73-5-6, May 1975.

9. Salotto, B.V., Grossman, E., and Farrell, J.B., "Elemental Analysis of Wastewater Sludges from 33 Wastewater Treatment Plants in the United States," in Pretreatment and Ultimate Disposal of Wastewater Solids, Rutgers University, May 21-22, 1974, EPA Report No. 902/9-74-002.

10. Fassell, M., "Sludge Disposal at a Profit?" Municipal Sludge Management, Barber-Colman Company, 1882 McGaw Ave., Irvine, California 92705.

EPA's RESEARCH PROGRAM IN SEWAGE SLUDGE COMBUSTION

R. A. Olexsey
Municipal Environmental Research Laboratory
Cincinnati, Ohio 45268

ABSTRACT

EPA's research program in the area of thermal decomposition of sewage sludges addresses the problems of increased sludge production, energy shortages, pollutant emissions, and cost. In the current fiscal year, EPA is supporting a number of projects in the areas of conventional incineration, starved-air incineration, and pyrolysis. The purpose, approach, and any significant results achieved to date are discussed for each of the major ongoing projects in the area of sludge combustion and pyrolysis. Pre-fiscal 1976 projects described include an engineering feasibility study on techniques for combined incineration of solid wastes and sludges, a full scale research investigation of incineration of sludge with coal, a pilot plant study of sewage sludge pyrolysis, and a full-scale demonstration of oxygen-enriched starved-air incineration. Projects tentatively planned for the fiscal 1976 and 1977 program years are outlined and briefly described.

INTRODUCTION

EPA's research program in sewage sludge combustion is directed toward the development of environmentally acceptable and cost effective alternatives to ocean disposal, which is no longer considered appropriate, and direct land disposal, which is not always economically or politically feasible. Table 1 outlines the projected growth of incineration as a sludge disposal medium based on current trends toward on-site disposal (1). Because of a number of external forces exerting pressures on both the sludge generation and sludge disposal operations development of effective thermal disposal techniques is not an easy task.

TABLE 1. TRENDS IN DISPOSAL OF SLUDGES (WEIGHT PERCENT)

	1972	1985
Disposal Methods		
Landfill	40	40
Utilized on Land	20	25
Incineration or Pyrolysis	25	35
Ocean (Dumping or Outfalls)	15	0

Foremost among the factors making matters difficult for sludge disposal is the fact that mandated Federal discharge standards require secondary treatment by 1977 and no pollutant discharge by 1985 (2). While the implementation of these standards will result in substantial improvements in water quality, an unfortunate by-product will be the production of much larger quantities of much wetter, more difficult to dewater sludge. Table 2 describes this expected growth in sludge generation (1).

TABLE 2. TRENDS IN PRODUCTION OF SLUDGES

	1972		1985	
Sludge Type	Pop. (Mill.)	Tons/Yr (Mill.)	Pop. (Mill.)	Tons/Yr (Mill.)
Primary	145	3.2	170	3.7
Secondary	101	1.5	170	2.5
Chemical	10	0.09	50	0.5
Totals		4.8		6.7

The production of more and wetter sludge impacts the second area of concern in sludge combustion; that is, the increasing cost and decreasing availability of the auxiliary fuel required to sustain combustion in sludge burners. Table 3 describes the effects of increased sludge cake moisture contents on auxiliary fuel demand in a multiple hearth furnace, with 800° F exhaust gas temperatures and 75 percent excess air (3). Thus, with current dewatering and combustion practices, the trend toward increased fuel consumption is directly counter to contemporary efforts at fuel conservation.

TABLE 3. ENERGY REQUIREMENTS FOR SLUDGE CAKES

Cake Moisture/ Solids Ratio	Lbs. Water Per Ton Dry Solids	BTU x 10^6 Required for Evap. of Water	BTU x 10^6 * Provided by Sludge Solids Ton	BTU x 10^6 Auxiliary BTU Requirement	Gallons of #2 Fuel Oil for Aux. Req. Per Ton
95/5	38,000	97.00	14.70	82.30	572
90/10	18,000	45.95	14.70	31.25	217
85/15	11,333	28.93	14.70	14.23	99
80/20	8,000	20.42	14.70	5.72	40
75/25	6,000	15.31	14.70	0.61	4
70/30	4,667	11.91	14.70	Autogenous	-
65/35	3,714	9.49	14.70	Autogenous	

* Assuming volatile solids of dry sludge solids = 70%
Heating value of volatile solids = 10,500 BTU/lb volatiles

Thirdly, air pollution emissions regulations and enhanced public sensitivity to air-quality and public health-environment interaction tend to create a somewhat hostile local atmosphere for incinerator siting. Although an EPA task force on sewage sludge incineration found that properly controlled well designed sludge incinerators were not a significant source of particulate emissions (4), attempts at incinerator installation are often met by vociferous public concern about such considerations as heavy metal emissions and microbiological aerosols.

Finally, the accelerating cost of incineration is the essential consideration in projecting the future for this option. The following listing notes that incineration is a high cost disposal technique relative to alternative means of disposal (1973 costs) (5).

Method	Total Costs ($/Dry Ton)
1. Disposal as liquid soil conditioner	20
2. Dewatered sludge as soil conditioner	32
3. Heat drying	64
4. Lagooning	15
5. Landfilling dewatered sludge	32
6. Barging to sea	15
7. Pipeline to sea	14
8. Incineration	32

If combustion is to remain a viable sludge elimination alternative, these issues of capacity, fuel, public health and cost must be squarely addressed. Toward this end, the Wastewater Research Division of the Municipal Environmental Research Laboratory is conducting and supporting a number of applied research, development, and demonstration projects in the areas of conventional incineration processes and advanced techniques such as pyrolysis and starved-air thermal degradation.

ONGOING RESEARCH

Currently, on a project by project basis, the research program in sludge combustion is divided roughly in half between efforts devoted to upgrading conventional incineration and tasks oriented toward development of new processes. Project formats range from paper design concept type studies to full-scale demonstrations of processes that have been proven successful in bench or pilot plant testing.

Review of Coincineration Technology

Sewage sludge incinerators demand supplemental fuel. Solid waste incinerators most often produce excess heat. Because of similarities in motivational circumstances, cities that practice solid waste incineration often also incinerate sewage sludge. Therefore, it would appear to be a perfectly logical development that the two materials be disposed of in a common incineration process. Conceptually at least, savings would result from decreased auxiliary fuel demand, capital economies of scale, and the need to operate only one facility. Figure 1 is an extremely simplified schematic of a combined incineration system. In this arrangement, hot flue gases from a solid waste incinerator are used to dry incoming sludge in a separate chamber. The dried sludge is then burned in the same furnace as the refuse.

In spite of the seemingly irrefutable logic of combined disposal, the history of coincineration in the U.S. had been a study in failure. Facilities enjoyed only short lives and were plagued by operational problems and high costs. In short, coincineration, while relatively successful in Europe, had simply not worked in the United States.

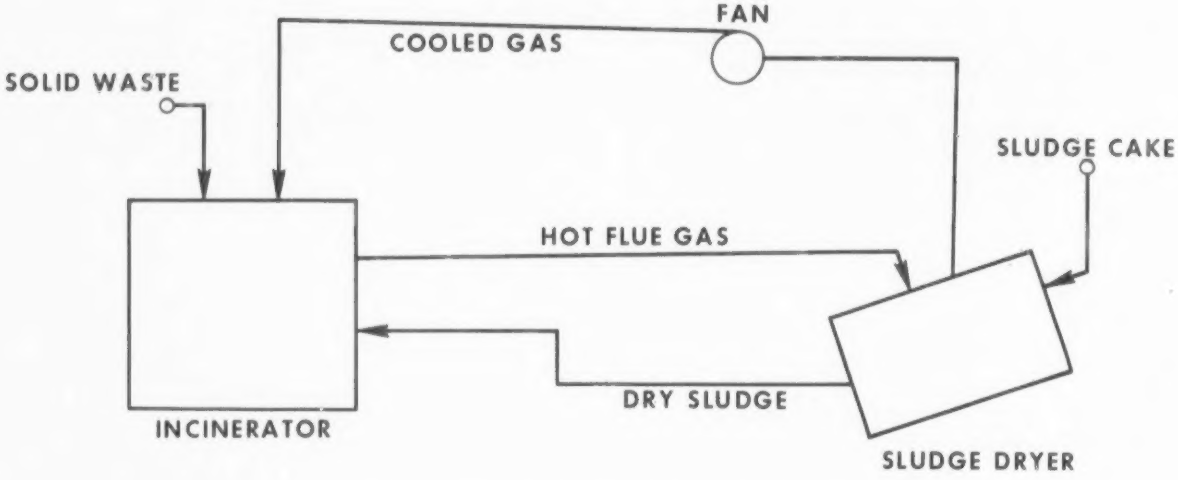


Figure 1. Simplified system for combined incineration.

To analyze the failure of coincineration and suggest feasible alternatives, the EPA, after competitive bidding, entered into a contractual arrangement with the Roy F. Weston Company of West Chester, Pennsylvania. Under the terms of this contract (EPA No. 68-03-0475), the contractor would explore the literature, visit operating sites in both the U.S. and Europe, and perform engineering feasibility studies of specific techniques.

While the final results of this study are not yet reported, some conclusions can be drawn. Coincineration was not approached seriously in the U.S. when fuel was abundant and cheap. Most coincineration facilities operated through simple addition of sludge cake to the feed of a solid waste incinerator. The high moisture contents of sewage sludges require design modifications to facilitate pre-drying of the sludge feed. Often, the refuse itself is too wet to burn without supplemental fuel.

Some successes have been achieved. A spray dryer-incinerator has operated intermittently at Ansonia, Connecticut, and a rotary dryer at Holyoke, Massachusetts, has performed reasonably well. As can be expected from any paper type study, starved-air incineration techniques ranked highly in the Weston evaluation.

The final report for the study will include design specifications for application of the most promising techniques to an existing sewage treatment plant. Cost data will be provided and local factors, such as geography and politics, which might impact implementation of a combined disposal system will be discussed.

Pulverized Coal as a Dewatering Aid

One solution to the fuel problem would be the production of a sludge cake feed that burns autogenously, that is, without auxiliary fuel. This, of course, can be accomplished by producing a cake that has a moisture content that is low enough so that the latent energy in the sludge solids is sufficient to evaporate the moisture in the cake. A second approach is to increase the volatile content of the cake to a point that the cake will support combustion.

Some previous EPA research work had shown that incinerator ash could be of significant value as a conditioning agent in vacuum filtration operations (6). Filter yield and cake solids content increased substantially with ash dosages in the range of from 1.0 lb. ash/lb. sludge solids to 4.0 lb. ash/lb. sludge solids.

While the addition of ash resulted in improved filter performance, it also reduced the energy content of the sludge cake because of the increased presence of inert material in the filter cake. An in-house research project was initiated to evaluate the performance of pulverized coal as a filtration aid. If the coal did not impair the function of the vacuum filter, then the fuel value realized from the coal addition could serve to allow the replacement of costly fuel oil with the less expensive coal.

Table 4 compares the properties of the coal and sludge used in the filtration study (7). The coal used was mine run Pennsylvania bituminous coal with a relatively low sulfur content. The sludge was a 1 to 1 by weight mixture of primary and waste activated sludge.

TABLE 4. COMPARISON OF THERMAL PROPERTIES

Property	Coal	Sludge
BTU/lb Dry Volatile Solids	14,200	10,500
Volatile Solids %	93.6	65-75
BTU/lb Dry Solids	13,300	7,370
Sulfur %	0.60	0.60

Figure 2 emphasizes the enhancement of sludge cake fuel value while the filter yield and final cake solids increased marginally with coal fines addition (7). The coal dosages monitored were in the range of from 0.1 lb. coal per lb. of sludge solids to 0.4 lb. coal per lb. of sludge solids.

Figure 3 describes the magnitude of coal addition required to achieve autogenous combustion conditions in a multiple hearth furnace at two significant exhaust gas temperatures (7). The range of addition of from 0.1 to 0.2 lb. coal per lb. of sludge solids that provides the needed energy represents significant economic savings over the conventional method of simple filtration and oil addition. At current prices, coal addition in the required range can accomplish cost reductions of from $4 to $15 per ton of dry sludge solids incinerated (3,7).

Further work is planned with coal and ash mixtures as filtration aids. The purpose of this work would be to reduce the usage of conditioning agents such as lime and ferric chloride.

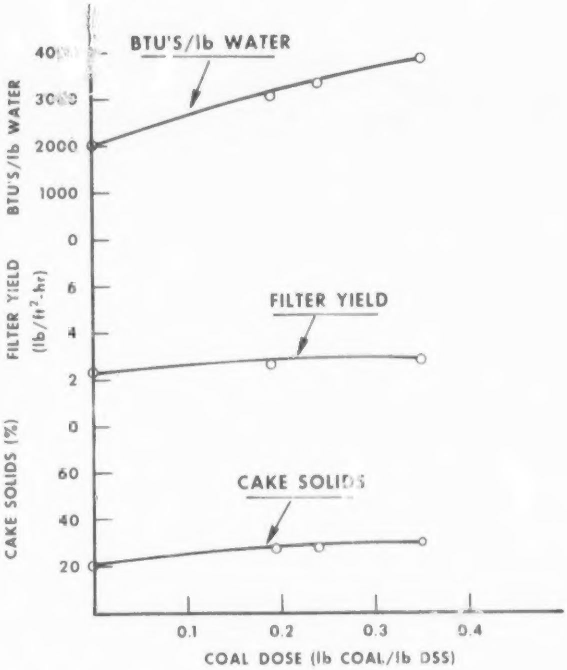


Figure 2. Coal performance.

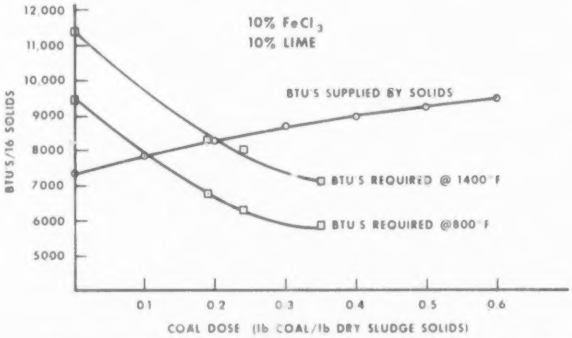


Figure 3. Thermal profile coal conditioned sludge.

Full Scale Incineration with Admixes

To underscore the threat that the fuel shortage poses to on-site sludge disposal systems, EPA recently entered into a research grant arrangement with the Metropolitan Waste Control Commission of the Twin Cities Area (Minneapolis-St. Paul). Under the terms of this grant (EPA No. R803927) a series of fuel admixes will be combusted with the sewage sludge feed to the six dry tons per day multiple hearth incinerators at the Commission's 24 mgd Seneca Sewage Treatment Plant. The plant is ideal in that the sludge disposal system consists of two independent and parallel lines, each including its own vacuum filter, conveyors, feed apparatus, and incinerator. One line will be used as a control and one line will be used to test the effects of the various admixes. The incinerators at Seneca resemble the model depicted in Figure 4.

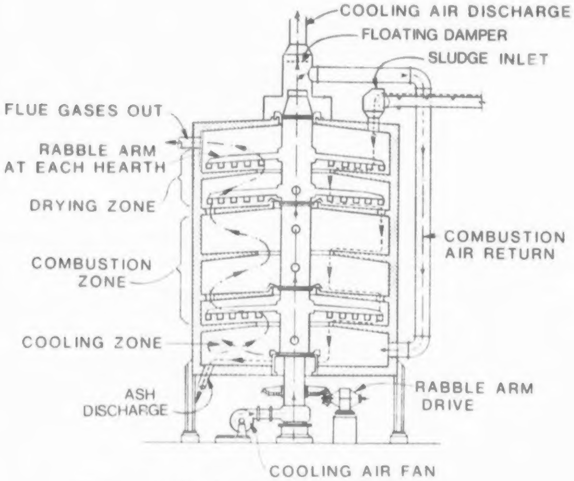


Figure 4. Typical section: Multiple Hearth Incinerator.

Tests of from one to four months' duration each will be conducted with various fuel additives in place of the normally used No. 2 fuel oil. Fuels will be lumped coal mixed with filter cake, pulverized coal mixed with liquid sludge prior to filtration, shredded and classified combustible solid waste, pelletized solid waste, scrap rubber tires, wood wastes, and combinations of fuels. Data collected during the course of the two-year project will include information on operational parameters such as mass balances of sludges and fuels, air flow rates, temperature profiles over time with changes in feed ratios, ash characteristics, and scrubber water and gas properties.

The grant is currently in the hardware procurement and facilities modification stage. Testing work will begin in the spring of 1976. The grant work is being cosponsored by the Wastewater Research Division and the Solid and Hazardous Wastes Research Division as part of its Wastes-as-Fuels Research program.

Pilot Plant Pyrolysis

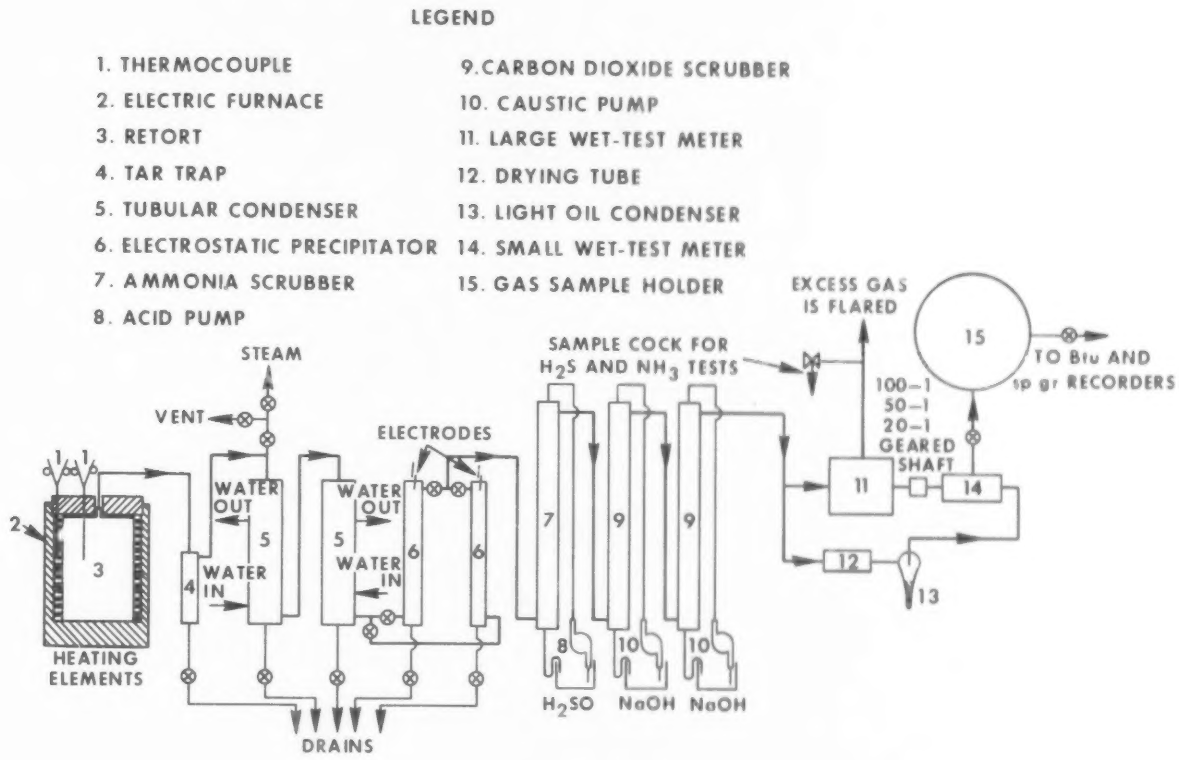
Pyrolysis is a process that holds much potential as a disposal technique for organic materials. Pyrolysis, which is the heating of a material in the absence of air, reforms organic materials into lower molecular weight compounds. The resultant products can be in the form of a gas, a liquid fraction, and a solid char, all with appreciable fuel values. Theoretically, air pollution is minimal, energy recovery capacity is high, and costs are roughly equivalent to those for incineration.

Pyrolysis had been proven effective on many organic waste materials, including municipal refuse (8). In order to explore the application of the concept of pyrolysis to sewage sludge disposal, EPA entered into an interagency agreement with the U.S. Bureau of Mines. Under the terms of this agreement (EPA No. IAG-D4-0436) primary sludge, activated sludge, sludge blends, and mixtures of sewage sludge and solid waste were pyrolyzed at the Bureau's Pittsburgh Energy Research Center pilot plant. A diagram of the pyrolysis pilot plant is presented in Figure 5. Wastes are fed into the batch retort and heated by an electric furnace. Tars and gases are cleaned and collected while the solid char remains in the retort.

Table 5 summarizes the yields of char, oil, and gas obtained from the pyrolysis of dried activated sludge at 500° C and 900° C. Pyrolysis of one ton of activated sludge at 900° produced approximately 1000 pounds of char, 13000 cubic feet of gas, and 30 gallons of oil. Total energy recovery was over 12.5 million BTU's per ton of feed.

Demonstration of Oxygen-Enriched Pyrolysis

Budgetary constraints are such that the funding of full-scale capital-intensive demonstration projects would absorb all of WRD's sludge disposal research funds. Therefore, in order to obtain maximum usage of the budgeted funds, attempts are made to involve EPA in limited testing of existing full-scale facilities. Such is the case with a demonstration grant awarded to the City of South Charleston, West Virginia. Under the terms of this grant (EPA No.



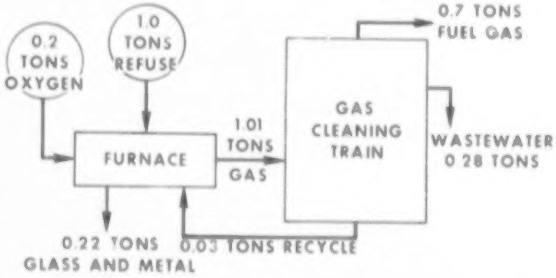
LEGEND
1. THERMOCOUPLE
2. ELECTRIC FURNACE
3. RETORT
4. TAR TRAP
5. TUBULAR CONDENSER
6. ELECTROSTATIC PRECIPITATOR
7. AMMONIA SCRUBBER
8. ACID PUMP
9. CARBON DIOXIDE SCRUBBER
10. CAUSTIC PUMP
11. LARGE WET-TEST METER
12. DRYING TUBE
13. LIGHT OIL CONDENSER
14. SMALL WET-TEST METER
15. GAS SAMPLE HOLDER
STEAM
VENT
ELECTRODES
WATER OUT
WATER IN
WATER OUT
WATER IN
HEATING ELEMENTS
DRAINS
SAMPLE COCK FOR H2S AND NH3 TESTS
EXCESS GAS IS FLARED
100–1
50–1
20–1
GEARED SHAFT
TO Btu AND sp gr RECORDERS
H2SO
NaOH
NaOH
1
1
2
3
4
5
5
6
6
7
8
9
9
10
10
11
12
13
14
15

Figure 5. Pilot Plant Pyrolysis System.

TABLE 5. SUMMARY OF YIELDS FROM PYROLYSIS OF DRIED ACTIVATED SLUDGE

Pyrolysis Temperature °C	500	900
Yields, Weight Percent of Feed		
Char	57.7	54.1
Gas	5.8	29.3
Tar, Oils, Aqueous	25.3	13.9
Yields, Per Ton of Feed		
Char, lb.	1154	1082
Gas, cu.ft.	2637	13415
Tar, Oils, Aqueous, gal.	57.7	29.6
Ammonium Sulfate, lb.	103.3	73.4
Energy, Million BTU/Ton of Feed		
Char	5.1	4.6
Gas	1.9	5.4
Tar, Oils, Aqueous	4.0	2.6

S803769) dewatered sewage sludge will be added to the feed to the 200 ton per day solid waste oxygen refuse conversion plant in that city. This plant, described schematically in Figure 6, was constructed to dispose of the municipal solid waste generated in South Charleston (9). The process was developed by the Union Carbide Corporation and has the trade name Purox.



0.2 TONS OXYGEN
1.0 TONS REFUSE
FURNACE
1.01 TONS GAS
GAS CLEANING TRAIN
0.7 TONS FUEL GAS
WASTEWATER 0.28 TONS
0.22 TONS GLASS AND METAL
0.03 TONS RECYCLE

Figure 6. Inputs and products of Purox System.

In the Purox system waste material is fed into the top and oxygen into the bottom of a shaft furnace. In the furnace there is a combustion zone where the oxygen is injected and a pyrolysis zone near the top. In the combustion zone, the inorganic fraction of the waste material is melted and exits as a slag. In the pyrolytic zone gases are produced from the organic fraction of the feed. The fuel gas produced has a heating value of about 300 BTU/SCF.

The grant work at South Charleston will be of limited duration with testing lasting only about four months. Through the course of the testing, optimum ratios of sludge to solid waste will be determined. The environmental impacts and economic feasibility of codisposal in the Purox system will be ascertained. Sludge addition will begin in the spring of 1976 upon completion of a solid waste reliability test.

PLANNED RESEARCH

Looking ahead to research that is generally planned for the next two years, we see that program emphasis remains on reducing cost and fuel consumption and on neutralizing the environmental effects of sludge combustion. Project mix will include feasibility, laboratory bench, and full scale demonstration projects.

In the next year, which is our fiscal year 1976, the most significant effort will be an operational study of the modifications required to convert an existing multiple hearth incinerator to pyrolytic operation. The widespread utilization of the multiple hearth furnace for sludge combustion indicates that the conversion approach may be the most direct method for rapid implementation of sludge pyrolysis.

Also to be funded in 1976 will be a laboratory study of the feasibility of steam reformation of liquid sludge and raw solid wastes in a rotary kiln reactor. Under this concept, the water from the wet sludge will serve as a steam medium for the conversion of the solid organics to a gas and an ash. The gas would then be cleaned, burned, and the spent exhaust used to power a turbine for electric power generation.

A third project in the next year will be an in-house study to characterize the char from sludge pyrolysis operations so that applications for this by-product might be found. A part of this study will be an investigation of the use of the char produced from pyrolysis of solid waste as a conditioning agent and fuel for sludge filtration and combustion.

Project planning for fiscal year 1977 can only be tentative. A mass and energy balance may be performed around an existing sludge incinerator so that the environmental effects of conventional incineration may be more completely described. The combined incineration process determined to be the most favorable through the ongoing Weston study may be demonstrated. Further studies of uses for carbon char from pyrolysis will be conducted, with the main emphasis being on the use of char as an activated carbon with the recovery of metals from the char as a by-product. Finally, the costs of sludge pyrolysis will be compared with the costs for conventional sludge incineration.

CONCLUSIONS

While it is impossible to design and fund a research program that encompasses all worthwhile endeavors, it is believed that the program described herein best utilizes available funds and best reflects foreseeable priorities. Future directions in combustion research will be dependent on any shifts in these priorities and on the magnitude of program funding.

REFERENCES

1. Farrell, J. B., "Overview of Sludge Handling and Disposal," in Municipal Sludge Management, Proceedings of the National Conference on Municipal Sludge Management, pp. 5-10, June 11, 1974.

2. Federal Water Pollution Control Act Amendments of 1972, Oct. 1972.

3. Hathaway, S. W., and Olexsey, R. A., "Improving the Fuel Value of Sewage Sludge," in News of Environmental Research in Cincinnati, Nov. 1975.

4. Environmental Protection Agency Task Force Report, "Sewage Sludge Incineration," Report No. EPA-R2-72-040 (Aug. 1972), NTIS PB 211323.

5. Olexsey, R. A., "Thermal Degradation of Sludges," in Pretreatment and Ultimate Disposal of Wastewater Solids, pp. 127-196, May 21, 1974, Report No. EPA-902/9-74-002.

6. Smith, J. E., Jr., Hathaway, S. W., Farrell, J. B., and Dean, R. B., "Sludge Conditioning with Incinerator Ash," Proceedings of the 27th Purdue Industrial Waste Conference, Eng.Ser. 141, Part 2, 911-925 (May 2-4, 1972).

7. Hathaway, S. W., and Olexsey, R. A., "Improving Vacuum Filtration and Incineration of Sewage Sludge by Addition of Powdered Coal," presented at 48th Annual Conference of the Water Pollution Control Federation, Oct. 9, 1975.

8. Sanner, W. S., Ortuglio, C., Walters, J. G., and Wolfson, D. E., "Conversion of Municipal and Industrial Refuse into Useful Materials by Pyrolysis," U. S. Bureau of Mines Investigation 7428, Aug. 1970.

9. "Solid Waste Disposal Resource Recovery," Union Carbide Bulletin F-3698.

URBAN STORMWATER MANAGEMENT AND TECHNOLOGY IN THE UNITED STATES - AN OVERVIEW

R. Field
Storm and Combined Sewer Section, WRD, MERL
Edison, New Jersey 08817

ABSTRACT

Combined sewer overflows are major sources of water pollution problems, but even discharges of stormwater alone can seriously affect water quality. Current approaches involve control of overflows, treatment, and combinations of the two. Control may involve maximizing treatment with existing facilities, control of infiltration and extraneous inflows, surface sanitation and management, as well as flow regulation and storage. A number of treatment methods have been evaluated including high rate screening and microstraining, ultra high rate filtration, dissolved air flotation, physical/chemical treatment, and modified biological processes. A swirl flow regulator/solids separator of annular shape construction with no moving parts has been highly developed. High rate disinfection methods including new disinfectants have been applied. Promising approaches involve integrated use of controls and treatment.

INTRODUCTION

Control and treatment of stormwater discharges and combined sewage overflows from urban areas are problems of increasing importance in the field of water quality management. Over the past decade much research effort has been expended and a large amount of data has been generated, primarily through the actions and support of the U.S. Environmental Protection Agency's Storm and Combined Sewer Research and Development Program. A summary which includes problem definition and management alternatives will be presented.

PROBLEM DEFINITION

The background of sewer construction lead to the present urban runoff problem. Early drainage plans made no provisions for storm flow pollutional impacts. Untreated overflows occur from storm events giving rise to the storm flow pollution problem.

Simply stated the problem is:

When a city takes a bath, what do you do with the dirty water?

Three types of discharges are involved: combined sewer overflows, storm drainage in separate systems, and overflows from infiltrated sanitary sewers. Significantly the storm path and collection system configuration may have a pronounced influence on combined overflow quality, resulting in simultaneous discharge mixtures of sewage and runoff at different points varying from raw to highly diluted as the system adjusts to a particular storm pattern. The problem constituents of general concern are visable matter, infectious bacteria, organics, and solids and in addition may include nutrients, heavy metals and pesticides. Representative comparisons are shown in Figure 1.

The average BOD concentration in combined sewer overflow is approximately one-half the raw sanitary sewage BOD. But storm discharges must be considered in terms of their shockloading effect due to their great magnitude. Urban runoff flowrates from an average storm intensity of 0.1 in/hr are 5 to 10 times greater than the dry-weather flow from the same area. Likewise a not uncommon rainfall intensity of 1 in/hr will produce flowrates 50 to

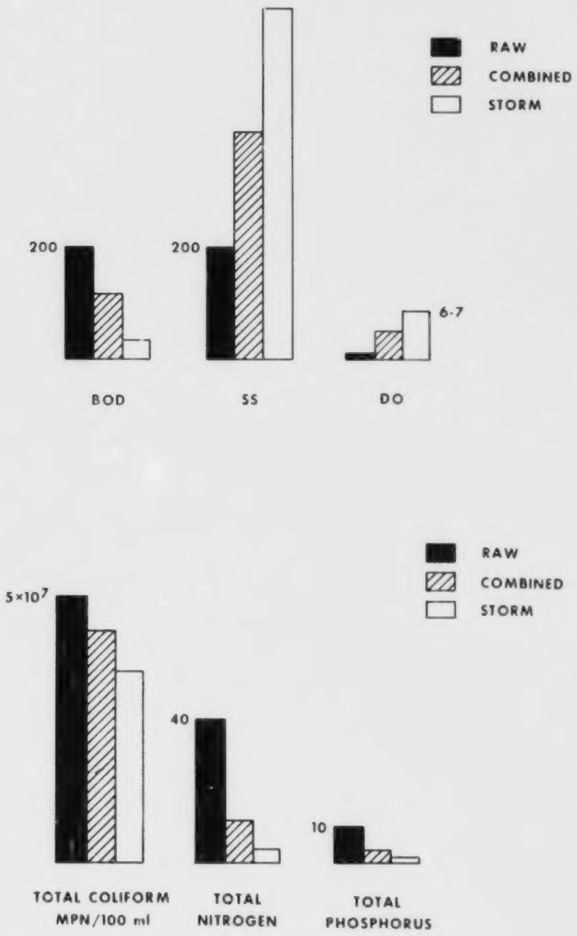


Figure 1. Representative Strengths of Wastewaters (Flow Weighted Means in mg/l)

100 times dry-weather flow. Even separate storm wastewaters are significant sources of pollution, "typically" characterized as having solids concentrations equal to or greater than those of untreated sanitary wastewater, and BOD concentrations approximately equal to those of secondary effluent. Bacterial contamination of separate storm wastewaters is typically 2 to 4 orders of magnitude less than that of untreated sanitary wastewaters. Significantly, however, it is 2 to 4 orders of magnitude greater than concentrations considered safe for water contact activities.

It is important to note that there is no apt description of "typical" combined sewage or stormwater runoff characteristics due to the variable nature of the rainfall-runoff patterns. Quality may range from super-strong sanitary sewage during the "first flush" to very diluted sewage later in the storm. The composition is dependent on a number of factors, including: length of antecedent dry weather, local climatic conditions, condition of the sewerage system and the nature of the drainage area.

A few municipal studies can serve to exemplify the problem. In Northampton, England it was found that the total mass of BOD emitted from combined sewer overflows over a two-year period was approximately equal to the mass of BOD emitted from the secondary plant effluent. The mass emission of suspended solids in combined sewer overflow was three times that of the secondary effluent.

The relatively poor flow characteristics of combined sewers during dry-weather when sanitary wastes alone are carried, encourages settling and build-up of solids in the lines until a surge of flow caused by a rainstorm purges the system. Studies in Buffalo, New York have shown that 20 to 30 percent of the annual collection of domestic sewage solids are settled and eventually discharged during storms. As a result, a large residual sanitary pollution load, over and above that normally carried is discharged over a relatively short interval of time, oftentimes resulting in what is known as a "first flush" phenomenon. This can produce shock loadings detrimental to receiving water life. Aside from the raw domestic and industrial sewage carried in the overflow, non-sanitary urban runoff in itself is a significant contributor to the overflow pollution load. As the storm runoff drains from urban land areas, it picks up accumulated debris, animal droppings, eroded soil, tire and vehicular exhaust residue, air pollution fallout, heavy metals, deicing compounds, pesticides and PCB's, fertilizers and other chemical additives, decayed vegetation, corrosion products, hazardous material spills, together with many other known and unknown pollutants.

A study on a 1.67 sq mi drainage area in Durham, North Carolina has shown that after providing secondary treatment of municipal wastes, the largest single source of pollution from the watershed is separate urban runoff without the sanitary constituent. Additional treatment of municipal waste could not be expected to significantly affect the total per acre yield of organic and suspended solids from the basin on an annual basis, though it might be necessary to protect the quality of watercourses during periods of dry weather and low stream flow.

When compared to the raw municipal waste generated within the study area the annual urban runoff of COD (as shown in Figure 2.) was equal to 91 percent of the raw sewage yield; the BOD yield was equal to 67 percent, and the suspended solids yield was 20 times that contained in the raw municipal wastes.

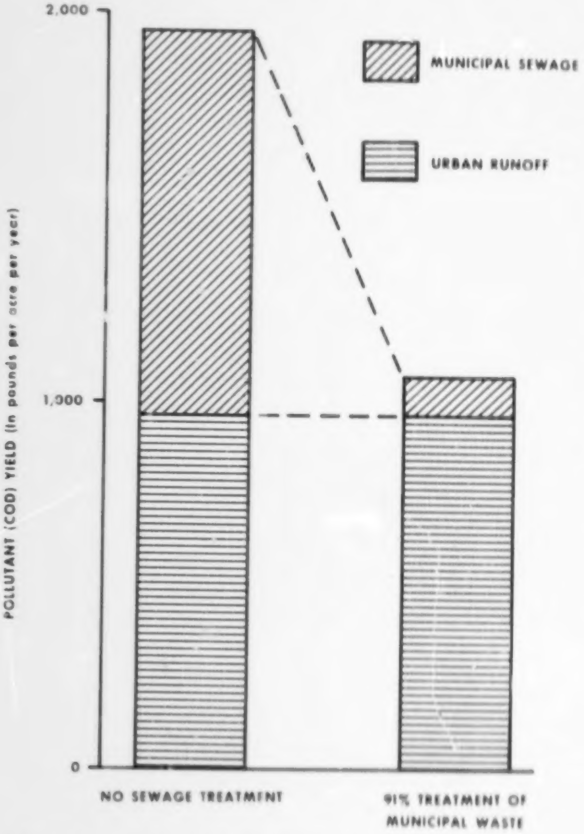


Figure 2. Annual pollutant (COD) yield from urban land runoff.

If Durham provided 100 percent removal of organics and suspended solids from the raw municipal waste on an annual basis, the total reduction of pollutants discharged to the receiving water would only be 52 percent of the COD, 59 percent of the ultimate BOD, and only 5 percent of the suspended solids.

During storm flows, dissolved oxygen content of the receiving watercourse was found to be independent of the degree of treatment of municipal wastes beyond secondary treatment. Oxygen sag estimates were unchanged even if the secondary plant was assumed upgraded to zero discharge, and stormwater discharges governed the oxygen sag 20 percent of the time.

Besides the aforementioned conditions in Durham, certain forms of solid waste such as beer cans, broken glass bottles, garbage, bed springs, shopping carts, etc., find their way into urban stream beds. These solid wastes, believed to be typical of urban streams, not only contribute to lower water quality, but are aesthetic pollutants adding to property devaluation and are a hazard to public safety as well. Figure 3. depicts coarse flotables from a combined sewer overflow in the Chicago, Illinois, USA area.

Figure 3. Coarse flotables from a combined sewer overflow, Chicago, IL, USA

DESIGN CONSTRAINTS

Precise characterization of the wastewater is virtually impossible because of the variability in the character of storm or combined wastewater and because of the many physical difficulties in representative sample collection. Also, because of the intermittency and variability of stormwater runoff and interrelated system flows, there is no such thing as an "average" design condition for storm flow treatment facilities. Therefore, a process that performs only when conditions are right or steady-state, may be too restrictive for practical applications.

CONTROL ALTERNATIVES

The concept of constructing new sanitary sewers to replace existing combined sewers has largely been abandoned due to enormous costs, limited abatement effectiveness, inconvenience to the public and extended time for implementation. It is again emphasized that urban stormwater runoff itself can be a significant source of stream pollution. Sewer separation would not cope with this pollution load. It is further estimated that the use of alternate measures could reduce costs to about one-third of the cost of separation.

What Can Be Done About The Problem?

The viable control alternatives are presented. First there is the problem at the source, e.g., at the land and streets, in the collection system, and off-line by storage. We can remove pollutants by treatment and by employing complex or integrated systems which combine various combinations of control and treatment including the dual-use of dry-weather facilities. Second, there is the choice of how much control or degree of treatment to introduce. Thirdly, there is the impact assessment, public exposure, and priority ranking with other needs. The proper management alternatives can only be made after cost-effective analysis involving goals, values, and hydrologic-physical system evaluations generally assisted by mathematical model simulations, pilot-scale trials, and new technology transfer.

Source Control

Source Control can be accomplished by employing porous pavement and upstream impoundment for flow attenuation; soil erosion preventative measures; restrictions on chemicals used for deicing, fertilization, pest control, and leaded gasoline; zoning and land use regulations; and improved neighborhood sanitation practices. The theory behind source controls is to limit the supply of contaminants. The benefits are not only reduced water pollution but also cleaner and healthier environments.

It is recommended that the newer and more promising street cleaning equipment such as vacuum sweepers, air brooms and wet scrubbers be further evaluated and employed as opposed to conventional sweeping and flushing methods. The newer devices offer benefits in picking up the urban runoff pollution causing dust and dirt particles rather than redistributing them for aesthetic purposes as the conventional devices do.

Sewerage System Control

In sewerage or collection system control the emphasis is on optimizing the existing collection system. Measures which can be used include:

- -Dry-weather flushing to reduce dry-weather solids accumulation and thereby relieving the overflow first flush.
- -Polymer feed to reduce overflows by increasing pipe carrying capacity.
- -Infiltration/Inflow prevention and correction.
- -And improved flow regulators or diversion devices, e.g., the swirl, helical and fluidic types.

A swirl flow regulator/solids-liquid separator being demonstrated in Syracuse, NY, USA is shown on Figure 4.

Figure 4. Swirl Regulator--Flotables entrapment during wet-weather operation, W. Newell St., Syracuse, NY, USA

Figure 5. is a schematic diagram. The device, of simple annular shape construction, requires no moving parts. It provides a dual function, regulating flow by a central circular weir while simultaneously treating combined wastewater by a "swirl" action which imparts liquid-solids separation. The low-flow concentrate is diverted via a bottom orifice to the sanitary sewerage system for subsequent treatment at the municipal works, and the relatively clear supernatant overflows the weir into a central downshaft and receives further treatment or is discharged to the stream. The developmental model in operation is shown in Figure 6. The device is capable of functioning efficiently over a wide range of 80:1 of combined sewer overflow rates, and can effectively separate suspended matter at a small fraction of the detention time required for conventional sedimentation or flotation (seconds to minutes as opposed to hours by conventional tanks). The capital cost of the 6.8 mgd Syracuse prototype was $55.000. O&M is estimated at $2,000/yr. Swirl construction cost curves are presented in Figure 7.

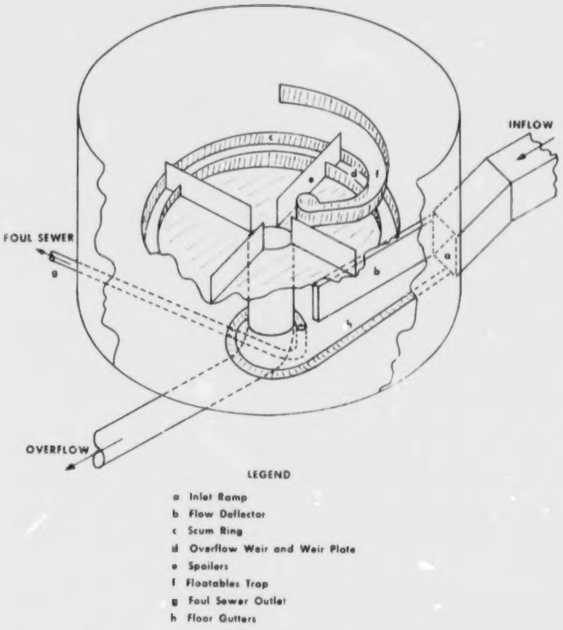


Figure 5. Isometric View of Swirl Regulator/Concentrator.

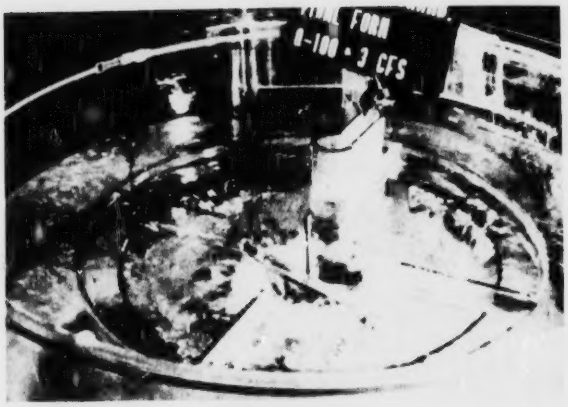


Figure 6. Overhead view of swirl regulator/separator in operation, Laboratory Hydraulic Model, LaSalle, Quebec, Canada

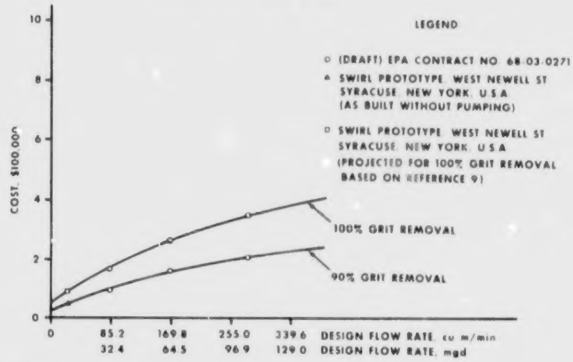


Figure 7. Estimated construction cost curves--swirl regulator/separator

Preliminary tests indicate at least 50 percent removal of suspended solids and BOD. Tables 1. and 2. contain further details on the Syracuse prototype treatability.

Figures 8. and 9. containing influent and effluent pollutographs show total suspended solids and BOD removed during a storm.

The swirl concept is also being piloted in Denver, Colorado and Toronto, Canada. At Denver it is being used as a degritter and at Toronto as a primary clarifier. This work is still ongoing, however preliminary test results are very encouraging. A helical or spiral-type regulator/separator has also been developed based on principles similar to those of the swirl device. This device is benefi-

Table 1. Suspended Solids Removal

	Swirl Concentrator						Conventional Regulator		
	Mass Loading kg			Average SS per storm, mg/l			Mass Loading kg		
Storm No.	Inf.	Eff.	% Rem.[b]	Inf.	Eff.	% Rem.[b]	Inf.	Underflow	% Rem.[a]
02-1974	374	179	52	535	345	36	374	101	27
03-1974	69	34	51	182	141	23	69	33	48
07-1974	93	61	34	110	90	18	93	20	22
10-1974	256	134	48	230	164	29	256	49	19
14-1974	99	57	42	159	123	23	99	26	26
01-1975	103	24	77	374	167	55	103	66	64
02-1975	463	167	64	342	202	41	463	170	34
06-1975	112	62	45	342	259	24	112	31	27
12-1975	250	168	33	291	232	20	250	48	19
14-1975	83	48	42	121	81	33	83	14	17
15-1975	117	21	82	115	55	52	117	72	61

[a] For the conventional regulator removal calculation, it is assumed that the SS concentration of the foul underflow equals the SS concentration of the inflow.

[b] Data reflecting negative SS removals at tail end of storms not included.

Table 2, BOD_5 REMOVAL

Storm No.	Mass Loading, kg Inf.	Eff.	% Rem.	Average BOD_5 per storm, mg/l Inf.	Eff.	% Rem.
7-1974	266	48	82	314	65	79
1-1975	97	30	69	165	112	32
2-1975	175	86	51	99	70	29

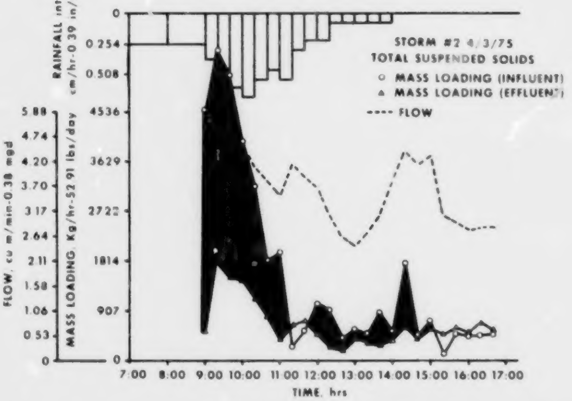


Figure 8. Swirl regulator suspended solids removal, W.Newell St., Syracuse, NY, USA

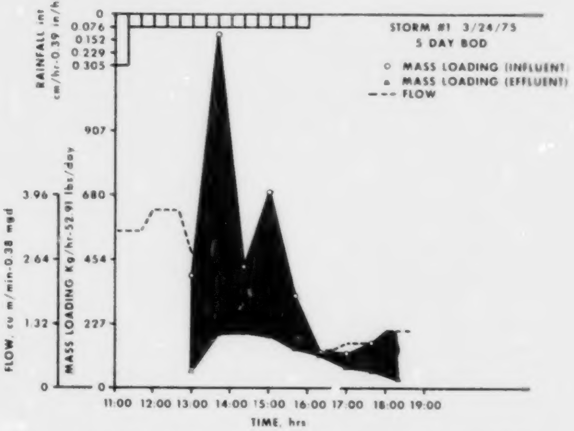


Figure 9. Swirl regulator BOD_5 removals, W. Newell St., Syracuse, NY, USA

cial as its solids separation action is created by only a bend in the sewer line.

Other collection system control methods are:

- In-sewer or in-line storage and routing whereby the intent is to assist a dispatcher in routing and storing storm flows to make maximum use of existing interceptors and sewer line capacity. The general approach comprises remote monitoring of rainfall, flow levels, and sometimes quality, at selected locations in the network, together with a centrally computerized console for positive regulation.

-And lastly, the most common approach is off-line or external storage with concrete tanks used most often. The concept is to capture wet-weather flow and bleed it back to the treatment plant during low flow dry-weather periods. The results of controlling overflow by detention are shown on Figure 10. Notice how an entire hypothetical overflow event at point A is prevented by storage with controlled dewatering.

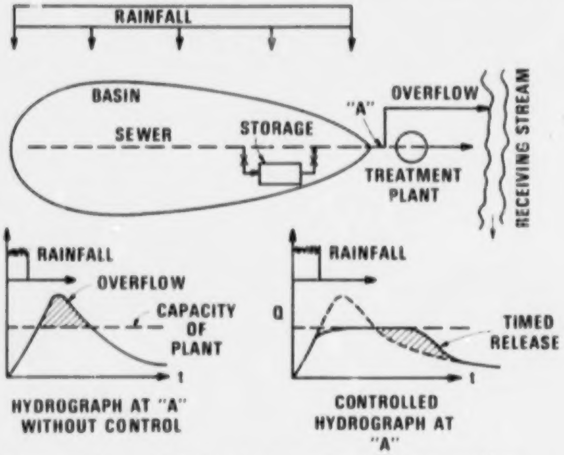


Figure 10. Results of controlling storm flow by storage

Storage facilities possess many of the favorable attributes desired in storm flow control: they are capable of providing flow equalization; are simple to design structurally, and operate; respond without difficulty to intermittent and random storm behavior; are relatively unaffected by flow and quality changes; and frequently can be operated in concert with regional dry-weather treatment facilities. Disadvantages of storage facilities include their large size, high cost, and dependency on other treatment facilities for dewatering and solids disposal. A 3.5 MG asphalt lined storage basin in Chippewa Falls, WI, USA eliminated 59 out of 62 river overflows during the evaluation period.

Treatment

The various treatment methods used for storm flow include physical and physical-chemical, biological, and disinfection. These processes or combinations of these processes, can be adjuncts to the existing sanitary plant or serve as remote satellite facilities at the outfall.

Physical and/or chemical treatment processes in many ways are well suited to storm flow applications, particularly with respect to solids removal because of their resistance to shockloads and rapid startup and shutdown characteristics. These processes include sedimentation, dissolved air flotation, screening, filtration, carbon adsorption and special swirl separation.

To reduce capital investments, demonstration projects have been directed towards operations approaching the maximum loading boundaries. Applications include their use for pretreatment or roughing, for the main or sole treatment, and, particularly in the case of microstrainers and filters, as effluent polishing devices.

The microstrainer is conventionally designed for polishing secondary sewage plant effluent at an optimum rate of approximately 10 gpm/sq ft. Tests on a pilot microscreening unit of 23 micron aperture in Philadelphia have shown that at high influx rates of 25-30 gpm/sq ft, suspended solids removals in combined overflows as high as 90% can be achieved. Since overflows are not continuous as sanitary flows are occurring about five percent of the total time, a sacrifice of screen life for increased hydraulic treatment rate is worthwhile.

A study in Cleveland showed high potential for treating combined sewer overflows by in-pipe filtration using anthrafilt and sand in a 7 to 8 foot bed. Figure 11. depicts the Cleveland pilot plant. With the high loadings of 16 to 32 gpm/sq ft surface area, removal of solids was effectively accomplished throughout the entire depth of filter column. Test work showed suspended solids removal up to

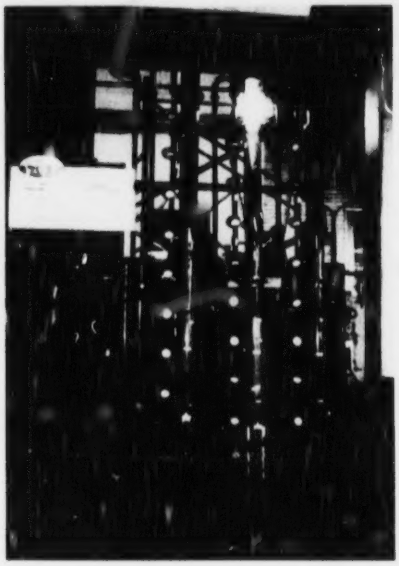


Figure 11. Pilot plant, Cleveland, OH, USA

and exceeding 90 percent and BOD removals in the range of 60 to 80 percent. Substantial reductions, in the order of 30 to 80 percent of phosphates, can also be obtained.

Results from a 5.0 mgd screening and dissolved-air flotation demonstration pilot plant in Milwaukee, WI, USA indicate that greater than 70 percent removals of BOD and suspended solids are possible. Findings also revealed 85 to 97 percent reduction in phosphate can be achieved as an additional benefit, by employing chemical coagulants.

Biological treatment of storm wastewaters must overcome some serious drawbacks: (1) the biomass used to assimilate the waste constituents must either be kept alive during times of dry weather; and (2) once developed, the biomass is highly susceptible to washout by hydraulic surges or overload.

Examples of biological treatment applications to stormwater include (1) the contact stabilization modification of activated sludge, (2) high rate trickling filtration, (3) bioadsorption using rotating biological contactors, and (4) oxidation lagoons of various types. The first three are operated conjunctively with dry-weather flow plants to supply the biomass and the fourth involves long term storage of the flows. With the exception of lagoons, some form of pre-unit flow equalization and control is essential for biological processes.

The most commonly used disinfectants under the research program are sodium hypochlorite, chlorine dioxide, and ozone. Because the disinfectant and contact demands are great in storm flow applications, due to intense flowrates, current research centers on high-rate applications (1) by imparting turbulence, (2) use of alternative, more rapid disinfectants, and (3) on-site generation of disinfectants. Successful results in all these areas have been demonstrated.

Integrated Systems

By far the most promising approach to urban stormwater management is the integrated use of control and treatment with an areawide multidisciplinary perspective.

When a single method is not likely to produce the best possible answers to a given pollution situation, various treatment and control measures--as previously described--may be combined for maximum flexibility and efficiency. One such combination might be: in-sewer or off-system storage for subsequent overflow treatment in specifically designed facilities, followed by groundwater recharge or recovery for water sports and aesthetic purposes. Another combination might be flow retention with pump or gravity bleed-back to the sanitary sewerage system.

In all cases the optimum abatement plan for stormwater overflow pollution will have to be evaluated separately for the geographical area in consideration. Aside from climatological conditions, terrain, and land uses, choice of control and treatment will depend on the existing sewerage system configuration. For example, separate systems with large contributory areas and few overflow points present problems and require design philosophies which differ from those of systems divided into many subdrainage areas with individual combined wastewater outfalls.

The temporary storage concept, previously discussed as a control process, also provides for a certain degree of treatment by settling, for excessive overflows greater than the design storage capacity discharging directly to the receiving stream. Likewise, this settling potential for flows less than design capacity, together with on-site solids disposal and/or controlled dewatering to the receiving stream (in accordance with assimilation capacity), which are usually overlooked, should be definitely considered.

Another approach in overcoming the extreme variation in overflow rates is to provide surge facilities prior to the storm flow treatment plant or the municipal plant. The surge basin(s) (or existing combined sewers) could furthermore serve a dual function in equalizing not only wet-weather flows but dry-weather flows as well. In this way, a single future treatment system can readily be designed for storm and sanitary flow conditions. This could also assist presently overloaded sanitary plants in obtaining more uniform operation. Short-term storage incorporated into the treatment plant would even out the daily cycle of dry-weather flows allowing for more efficient use of the treatment process over the entire 24 hours. Equalization would permit reduced treatment process design capacity. Further analysis is necessary to determine the most economical break-even point between the amount of storage versus the treatment capacity. The designer should recognize the wet-weather treatment plant's capability to draft stored flow continuously while it is raining in his evaluation of the optimum surge-treatment system. Dual-use wet-weather storage and treatment facilities built in conjunction with dry-weather plants can be used to improve dry-weather treatment capacity and effectiveness the vast majority of the time when it is not raining.

The program has fostered various schemes which reclaim storm flows for beneficial purposes including the enhancement of visual aesthetics, recreation, and water supply.

Mount Clemens, MI, USA has installed a treatment-park system involving discharge of combined sewage overflows into a series of three "lakelets" each equipped with surface aerators (schematic, Figure 12.). Effluents pass from one pond to the next through microstrainers and filters, and the final effluent is chlorinated. This control and treatment scheme is designed to have no adverse aesthetic impacts and blend into a surrounding park development and the waters are being reclaimed for recreation and reused for park irrigation.

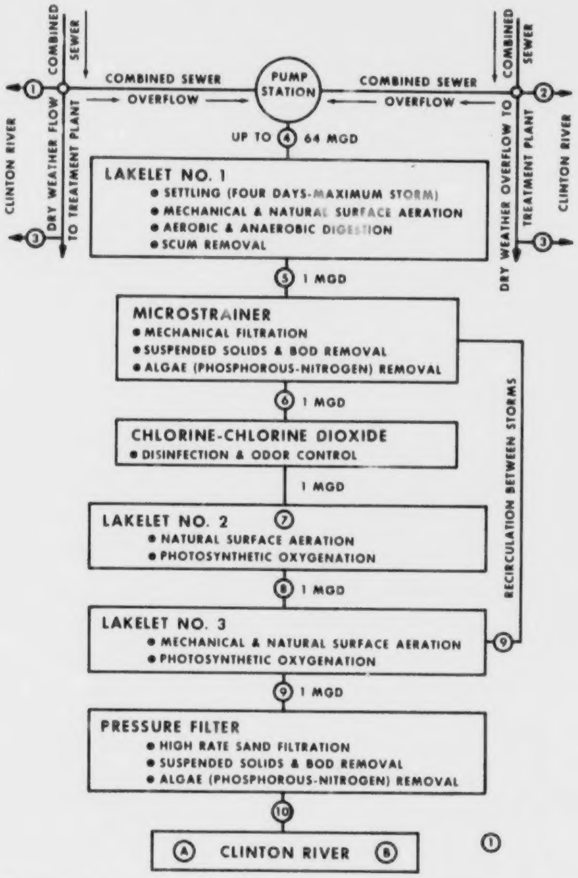

COMBINED SEWER
COMBINED SEWER
OVERFLOW
PUMP STATION
COMBINED SEWER
OVERFLOW
COMBINED SEWER
①
②
CLINTON RIVER
DRY WEATHER FLOW TO TREATMENT PLANT
③
UP TO ④ 64 MGD
DRY WEATHER OVERFLOW TO TREATMENT PLANT
CLINTON RIVER
③
LAKELET NO. 1
● SETTLING (FOUR DAYS-MAXIMUM STORM)
● MECHANICAL & NATURAL SURFACE AERATION
● AEROBIC & ANAEROBIC DIGESTION
● SCUM REMOVAL
⑤ 1 MGD
MICROSTRAINER
● MECHANICAL FILTRATION
● SUSPENDED SOLIDS & BOD REMOVAL
● ALGAE (PHOSPHOROUS-NITROGEN) REMOVAL
⑥ 1 MGD
CHLORINE-CHLORINE DIOXIDE
● DISINFECTION & ODOR CONTROL
1 MGD
LAKELET NO. 2 ⑦
● NATURAL SURFACE AERATION
● PHOTOSYNTHETIC OXYGENATION
⑧ 1 MGD
RECIRCULATION BETWEEN STORMS
LAKELET NO. 3
● MECHANICAL & NATURAL SURFACE AERATION
● PHOTOSYNTHETIC OXYGENATION
⑨
⑨ 1 MGD
PRESSURE FILTER
● HIGH RATE SAND FILTRATION
● SUSPENDED SOLIDS & BOD REMOVAL
● ALGAE (PHOSPHOROUS-NITROGEN) REMOVAL
⑩
Ⓐ CLINTON RIVER Ⓑ
①

Figure 12. Schematic diagram of Mount Clemens, MI, USA facility.

Computer Assistance

Mathematical models are needed to predict complex dynamic system responses to variable and stochastic climatological phenomena.

These models have been developed and applied at many levels of sophistication including EPA's Storm Water Management Model (SWMM) which is capable of representing the gamut of urban runoff conditions both qualitatively and quantitatively from the uppermost catchment point to the downstream receiving water.

RESULTS AND COSTS

Detroit found the cost of in-system controlled storage to be as low as $0.02 to $0.04/gal. This range was approximately one-tenth the estimated cost for large off-line facilities.

Typical abatement system construction costs are summarized based on mid-1973 prices.

Table 3. TYPICAL COSTS OF COMPONENT DEVICES AND OPERATIONS, ENR 2000

In-line storage	$0.02-0.25/gal
Off-line storage	$0.20-4.75/gal
*Physical treatment	$5,000-35,000/mgd
*Physical/chemical treatment	$35,000-80,000/mgd
*Biological treatment	$60,000-80,000/mgd

*for a hypothetical 25 mgd plant

These data are based on a very limited number of specific projects; thus, they represent only an order of magnitude placement. In extrapolating these costs into master plan systems, such as the City of San Francisco's (CA, USA), the totals frequently approach $500-1,000/acre. It is estimated that the national average per acre cost for sewer separation would be $20,000-$30,000 today. Whereas a similar estimate for the control alternatives would fall in the neighborhood of $10,000/acre.

A simplified example will serve to illustrate an advantage of integrated approaches. Assume a design composite storm overflow for a combined catchment area is three hours long and the hydrograph (Figure 13.a.) is triangular shaped with a maximum value of 25 mgd occurring at the end of the first hour. Total containment in storage would therefore require a capacity of 1.56 MG which at a unit cost of $0.50/gal would cost $780,000 to construct. Similarly a treatment facility designed for the maximum flowrate might cost $750,000 at $30,000/mgd. A 10 mgd plant, however, coupled with a storage capability of 0.56 MG would accomplish the same objectives for $580,000 as illustrated by Figure 13.b. Of particular importance is the opportunity for the treatment plant to operate at its design capacity for a sustained period of time.

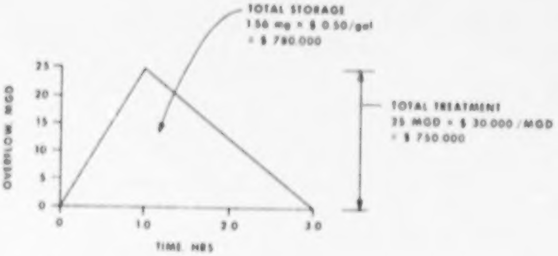


Figure 13. (a) Separate Approach
Storage or Treatment

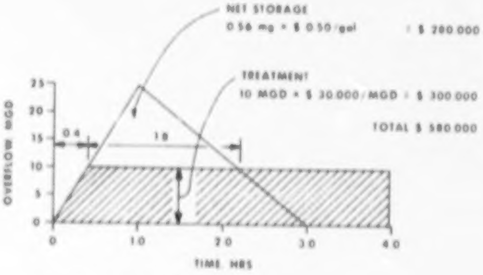


Figure 13. (b) Integrated Approach
Storage and Treatment

While prototype installations in operation today are still relatively few in number, some impressive results have been obtained. The in-system storage concepts have proven feasible to operate and maintain with a major curtailment of overflow occurrences and durations. Off-line storage and detention-chlorination facilities are proven performers with additional benefits for backing up overloaded treatment works innovated. Storm flow facilities constructed in parallel with conventional dry-weather flow treatment plants have introduced dual-use functions improving even dry-weather performance as well as increasing treatment capacities during storm periods.

CONCLUSIONS

All facts point to a real requirement for treating and controlling stormwater runoff and combined sewer overflows. In view of the tremendous quantities of pollutants bypassed during rainfall from the combined sewer system, it does not seem reasonable to debate whether secondary treatment plants should be designed for 80, 85 or 90% BOD or suspended solids removal, when in fact the small increments gained in this range are completely overshadowed by the bypassing occurring at regulators during wet-weather flow. During a single storm event, from 95% to 99% of the total organic load is attributed to runoff generated discharges. On an annual basis the storm associated organic load may be from 40% to 80% the total figure. It has been estimated that detrimental wet-weather receiving stream impacts can last for days.

It is a distinct possibility that communities may make expensive sewage treatment plant improvements and still not achieve water quality goals due to the impact of urban runoff (or some other nonpoint source) unless steps are taken to harmonize land use with water quality or treat the runoff waters to reduce their adverse effects on receiving streams.

Along with planning to upgrade secondary sewage treatment plants, and because of possible contravention of stream standards, we should carefully assess the potential impact of urban runoff.

The multi-billion dollar treatment plant upgrading and expansion program now going on throughout the country will do much to alleviate pollution of our waters. However, means of mitigating the effects of combined sewers and stormwater must also be found if we hope to abate the pollution in an optimal manner. Wet-weather standards continue to be promulgated.

ACTIVATED CARBON FOR MUNICIPAL WASTEWATER TREATMENT

James J. Westrick
Municipal Environmental Research Laboratory
Cincinnati, Ohio 45268

ABSTRACT

Full scale use of activated carbon adsorption as a municipal wastewater treatment process has resulted in part from the early vigorous research efforts of EPA's predecessor organizations. Research is continuing in pilot plant operations in Cincinnati and elsewhere to further understand the process and its implications in the water pollution control effort. Completed and ongoing research projects dealing with influence of chemical pretreatment, pH, nitrate addition, and breakpoint chlorination are discussed. The removal of general organic pollutants, heavy metals and specific organics such as halomethanes, is described with regard to the studies conducted or sponsored by EPA. A brief description of the independent physical-chemical system at Rocky River, Ohio, and the performance and cost evaluation study of the completed facility are included.

INTRODUCTION

The use of activated carbon for municipal wastewater treatment was one of the principal results of the research sponsored in the early days of the Advanced Waste Treatment group of the U.S. Public Health Service. Research at Harvard, Pittsburgh, Cincinnati, Lebanon, Pomona and Tahoe established the feasibility of utilizing granular activated carbon beds for the removal of organic matter from secondary effluents and regenerating spent carbon for reuse. The first full-scale application of this research was the 7.5 mgd (28,000 cu m/day) advanced waste treatment facility at South Tahoe, California. Here activated carbon was utilized as the last treatment step in an extensive train of processes beginning with conventional activated sludge through two-stage lime clarification, ammonia stripping, filtration and finally, adsorption. Lime sludge recalcination and carbon regeneration were also incorporated. This pioneering facility has been on stream since 1968, producing high quality water for irrigation and recreational uses.

Encouraged by the success at Tahoe and assisted by Research Development and Demonstration Grant funding, several other local authorities constructed tertiary carbon systems at Colorado Springs, Colorado, Nassau County, New York, and Piscataway, Maryland.

A number of studies by private interests and research sponsored by EPA's predecessor organizations opened the possibility of using carbon as an alternative to biological systems for secondary treatment. Activated carbon was shown to be effective for the production of high quality effluent when preceded by efficient chemical clarification. A flurry of feasibility studies followed and several municipalities opted for systems consisting of chemical clarification of raw wastewater followed by activated carbon beds. Such systems, called Physical-Chemical (P-C) Systems, promised some apparent advantages when compared to conventional secondary treatment systems. These are shown in Table 1. The space savings resulted in the selection of P-C treatment at Rocky River and Cleveland Westerly. It is especially dramatic at Cleveland, where a

50 mgd (190,000 cu m/day) P-C system will be constructed on a 7-acre (3 ha) site, with room for future expansion. Other municipalities chose P-C to best cope with local conditions, such as unusual raw wastewater characteristics or high quality effluent requirements.

TABLE 1 - FEATURES OF P-C TREATMENT

1. Minimum land requirements
2. Low sensitivity
3. Unaffected by toxic materials
4. Flexibility of design and operation
5. High organic removal
6. High phosphorus removal
7. Capability for heavy metal removed

Because of the relatively high purchase price of granular activated carbon, regeneration and reuse of spent carbon is commonly practiced. All municipal regeneration systems currently in use or planned utilize thermal regeneration systems wherein spent carbon is raised to a temperature of ~900°C-1000°C by contact with hot gases of combustion in a multiple hearth furnace. Steam is normally injected to oxidize the adsorbed organics.

Whether on-site regeneration is economic depends upon carbon utilization rate, the plant size, the delivered cost of virgin carbon, local labor cost, and installed cost of regeneration equipment. On-site regeneration in multiple hearth furnaces is not feasible at very small plants, because the smallest fully automated commercially available multiple hearth regeneration furnace has a firm capacity of 3000 lb/day (1400 kg/day). Other types of thermal regeneration systems may be developed in the future which will allow economic regeneration of spent carbon at small facilities. One example is a unit which uses infrared lamps to heat carbon passing through a controlled atmosphere on a continuous conveyor. Rotary kilns might also be economic at the small plant size.

The demonstrated success of granular carbon regeneration both at Tahoe and in the sugar industry and uncertainty as to the feasibility of regenerating powdered carbon (mean particle size ~ 10-20 μ) favored the development of granular carbon over powdered carbon systems for municipal wastewater applications. Powdered carbon was shown to be effective both in tertiary and P-C applications in a series of pilot studies sponsored by EPA and predecessor organizations. Pilot scale regeneration has been successful at several locations, and in fact, a 10 T/day (9000 kg/day) powdered carbon regeneration system is now operated by a carbon manufacturer to regenerate spent sugar carbon. Thus, renewed interest in powdered carbon for municipal use is expected.

CINCINNATI PILOT PLANT ACTIVITIES

Several recent projects have been conducted at the Cincinnati EPA facility to learn more about the use of activated carbon for municipal wastewater treatment.

Effects of Chemical Pretreatment on Carbon Adsorption (1)

As suggested earlier, when carbon is used in the P-C system (no biological treatment), it is normally preceded by chemical clarification and sometimes filtration. The chemicals used are those which also precipitate phosphorus, namely lime or salts of aluminum or iron. The chemical clarification systems have generally been selected on the assumption that the type of chemical pretreatment used would have no effect upon the subsequent adsorption of organics by activated carbon. However, several reports were published which claimed that high pH lime clarification caused alkaline hydrolysis of high molecular weight organic molecules and produced a subsequent beneficial effect on carbon adsorption in terms of effluent quality and carbon capacity. In order to investigate the influence of chemical pretreatment on carbon performance, three P-C pilot systems were constructed, operated with three distinctly different chemical clarification schemes. Data were collected and evaluated with special emphasis given to carbon performance.

The three chemical treatments used were

1. High lime - 690 mg $Ca(OH)_2$/l, pH>11.5 neutralized to pH<7 by CO_2+H_2SO_4
2. Low lime - 320 mg $Ca(OH)_2$/l+15 mg Fe^{3+}/l, pH ~ 10
3. Iron - 62 mg Fe^{3+}/l, pH 6.0-6.5

A schematic of the iron system is shown in Figure 1. The lime systems were similar with the addition of retention and neutralization tanks.

Each chemical clarifier was followed by a dual-media (anthracite-sand) filter and a carbon adsorber. Each adsorber was made up of 5 - 4-in-(10 cm) diameter tubes in series with a total depth of 18 ft (5.5 m) of Filtrasorb 300 granular carbon. The application rate was 4 gpm/sq ft (10 m/hr) for a total empty bed contact time of 34 min. The systems were operated continuously for eleven months except for brief shutdowns for minor mechanical failures.

The overall performance of the three systems is presented in Table 2. The source of raw sewage for this project was a 30-in-(76 cm) diameter interceptor sewer serving a residential-commercial neighborhood. The sewage could be described as weak, partly because of infiltration during periods of rainfall. There were some differences in performance of the three systems based on the removal of the

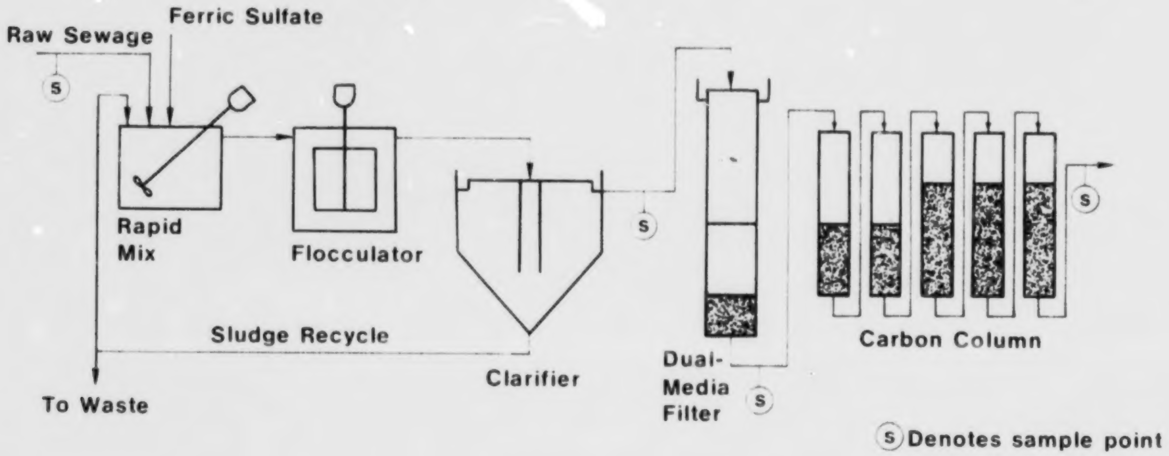


Figure 1. Flow diagram, iron system.

Table 2. Average performance of P-C Pilot System.

Item	TOC		COD		TSS		P		Alk. (mg/l as $CaCO_3$)	Turb. (JTU)	Iron (mg/l)	Color Units	Ca (mg/l as $CaCO_3$)	Mg (mg/l as $CaCO_3$)
	mg/l	% Removal	mg/l	% Removal	mg/l	% Removal	mg/l	% Removal						
Raw Sewage	59.9		215		98		9.2		209	38	0.7	-	182	65
Low-Lime System:														
Clarifier Eff.	17.4	71	52.0	76	20	80	1.3	86	165	11	1.3	-	200	46
Filter Eff.	12.5*	79	36.9*	83	7	93	0.9	90	156	5	0.4	16	190	43
Carbon Eff.	7.1*	88	22.8*	89	6	94	0.9	90	165	11	0.7	7	204	32
Iron System:														
Clarifier Eff.	14.1	76	42.2	80	28	71	1.0	89	54	21	8.2	-	-	-
Filter Eff.	9.5*	84	27.8	87	12	88	0.3	97	62	13	3.5	10	-	-
Carbon Eff.	4.9*	92	17.0*	92	7	93	0.3	97	66	12	2.9	5	-	-
High-Lime System:														
Clarifier Eff.	15.9	73	43.0	80	17	83	0.5	95	209	8	-	-	410	7
Filter Eff.	12.9*	78	36.5*	83	9	91	0.5	95	200	4	-	14	385	9
Carbon Eff.	6.2*	90	20.8*	90	8	92	0.5	95	163	7	-	5	389	8

*Flow-weighted averages. All other values are arithmetic means major pollution parameters. The iron system generally produced the lowest residuals in terms of organics and phosphorus while the residual suspended solids in the effluents were essentially equal. The carbon systems were run to exhaustion; thus the organic concentrations in the effluents are the averages of values ranging from very low concentrations with fresh carbon to essentially complete breakthrough.

Figure 2 illustrates the decline in performance of the carbon systems with cumulative applied loading. It is apparent that the removal of TOC by the iron-carbon adsorber declined at a greater rate than the lime systems. This is to be expected since the iron clarified-filtered wastewater had a lower concentration of TOC than the other streams. According to accepted adsorption models, organic removal capacity of carbon decreases as the concentration of organics in the wastewater decreases. The high lime system performed somewhat better than the low lime system.

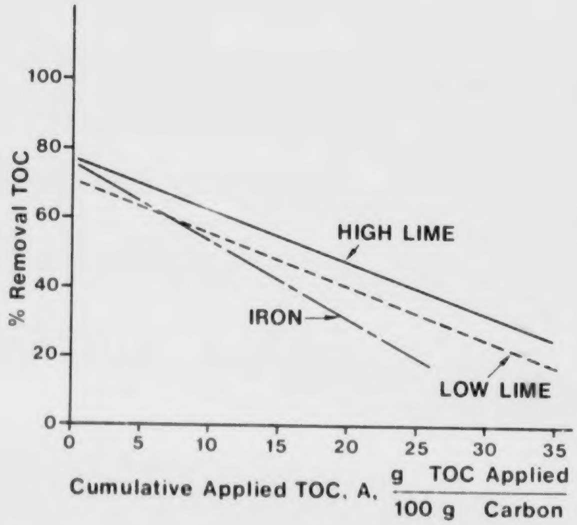


Figure 2. Performance of activated carbon systems.

In Figure 3 the breakthrough of TOC is shown as a function of specific throughput or liters of sewage passed through the adsorber per gram of carbon in the adsorber. Also shown on the abscissa is the single-stage carbon dosage corresponding to the specific throughput.

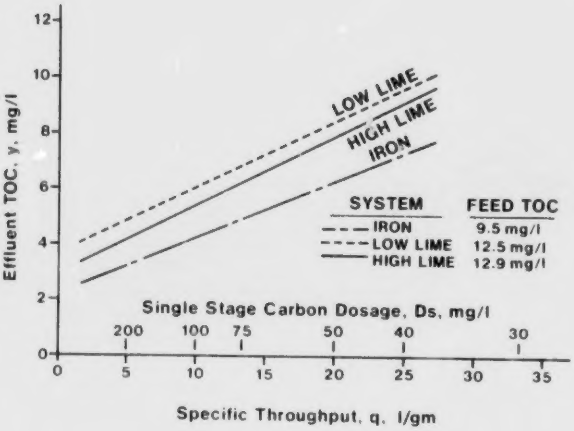


Figure 3. Idealized breakthrough curve.

If, at a given specific throughput, the carbon in a single adsorber is considered exhausted, it would be removed and replaced, and the carbon dosage at that point would be calculated as the reciprocal of the throughput in appropriate units. Figure 3 shows that for any given effluent standard the iron system will be able to treat more sewage and the carbon dosage will be less than for the other systems.

A single carbon contactor treating an entire waste stream is the least efficient contacting system from a standpoint of carbon loading. The entire contents of the single-stage contactor must be removed and replaced with virgin or regenerated carbon when the effluent reaches the specified limit. Only a fraction of the carbon in the bed is completely saturated while a large portion of the bed is in a state of partial exhaustion. Other methods of contacting are commonly used, such as multiple-staged series operation, pulsed bed operation and multiple single-stage parallel operation. Of these, only the multiple single-stage parallel system can be applied directly to the data obtained in this project, since these adsorbers were strictly single stage.

The carbon utilization in single-stage contacting can be improved by dividing the total contactor volume requirement into a number of parallel contactors. By starting the contactors in staggered sequence, it is possible to have on-stream at any given time contactors in various stages of exhaustion. Thus, effluents from more heavily loaded contactors can be blended

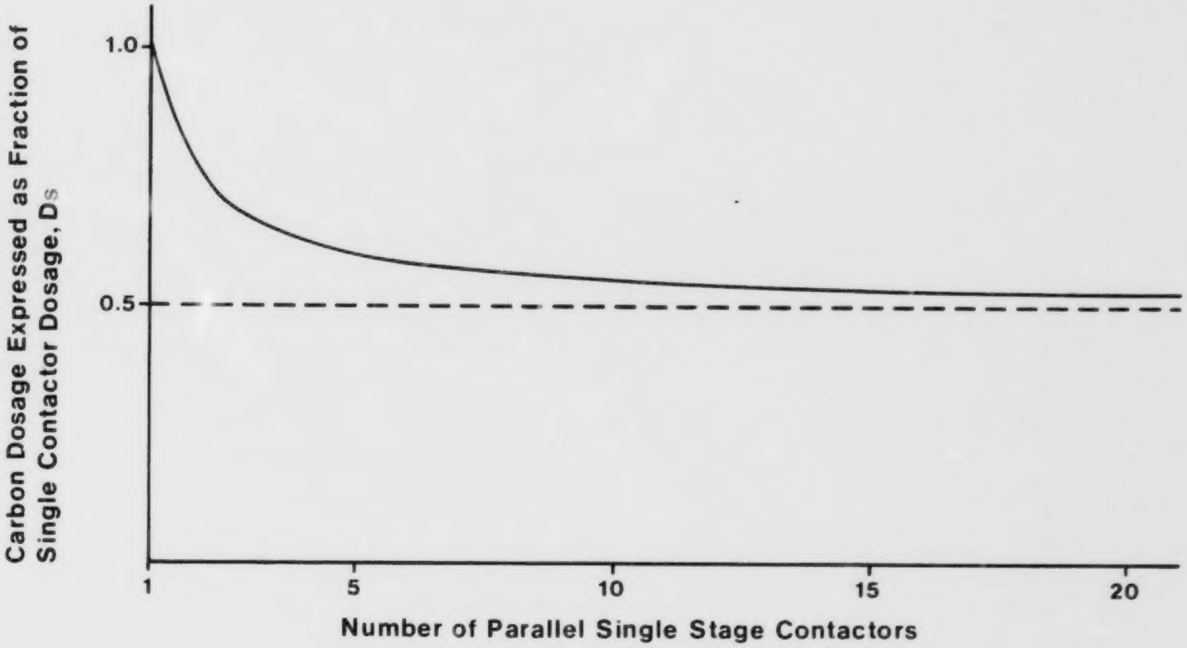


Figure 4. Effect of multiple contractors on carbon dosage.

with effluents from less heavily loaded contactors to produce the desired effluent quality.

Figure 4 shows the savings in carbon as the number of parallel contactors increases. For example, the carbon dosage for an eight-contactor system with staggered start-up is 56% of the single contactor dosage.

To provide additional information for the performance comparison, cost estimates for treatment by carbon systems that perform in accordance with the data obtained in this study were made. These estimates include amortization of capital as well as operation and maintenance costs. The assumptions used in the estimate are shown in Table 3. The estimates were made for a 10 mgd (38,000 cu m/day) carbon adsorption system with eight parallel, single-stage contactors at 34 min contact time. The relationship of carbon treatment cost to carbon dosage was computed and is shown in Figure 5. The treatment cost increases with carbon dosage; nearly all of the increased cost results from regeneration operation and maintenance cost, including carbon makeup.

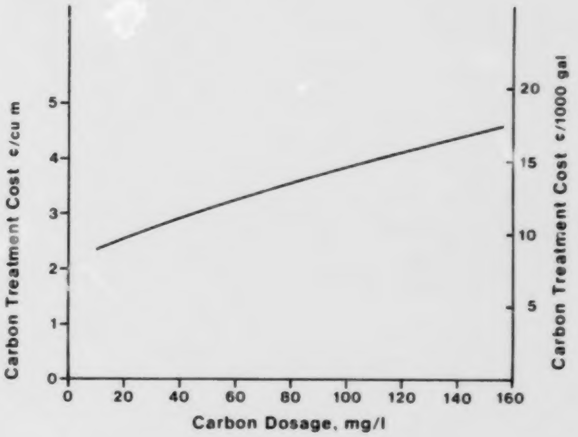


Figure 5. Relationship of carbon treatment cost to carbon dosage.

Figure 6 shows the effect of the three pretreatment systems on the cost of carbon treatment for varying effluent TOC levels. Differences in carbon treatment cost for the three systems studied are significant at the low effluent TOC levels, but they diminish as the required level of treatment is reduced. Over the entire range of effluent TOC considered in this

Table 3 - Input Assumptions for Carbon System Cost Estimation

Item	Input Assumptions
Avg. daily flow	10 mgd(38,000 cu m/day)
Peak flow	14.8 mgd (56,000 cu m/day)
Amortization interest rate	6.0%
Amortization period	25 yr
STP CCI (EPA)*	2.025
Wholesale price index*	1.535
Hydraulic surface loading	4 gpm/sq ft(10 m/hr)
Carbon contact time	34 min
No. of contactors	8
Carbon cost	\$0.42/lb (\$0.93/kg)
Regeneration loss	8%
Power cost	\$0.020/kw hr
Fuel cost	\$0.10/therm(\$0.0006/MJ)
Labor wage rate	\$5.50/hr

*June, 1974.

study, the lowest feed TOC provided by the iron pretreatment system permits the least cost carbon treatment. High-lime pretreatment results in lower cost carbon treatment than low-lime pretreatment with the advantage diminishing greatly at higher allowable effluent TOC levels.

These carbon treatment cost comparisons can thus be incorporated into total system cost comparisons to provide a decision making mechanism for selecting the least total cost system. Such additional parameters as clarification area requirements, sludge dewatering requirements, neutralization requirements, complexity of operation and maintenance, influent quality characteristics, and effluent quality requirements should all be taken into account in selecting the chemical pretreatment system. Figure 6 suggests that differences in carbon treatment cost that would result from utilizing the various chemical pretreatments studied here are large enough at low effluent TOC levels to be considered in making the selection of chemicals. For example, if the effluent TOC requirement were 5 mg/l, the low-lime pretreatment system, including clarification, neutralization, filtration, and sludge handling, would have to cost at least 3.4¢/1,000 gal (0.9¢/cu m) less than a pretreatment system using iron. Likewise, a high-lime system (which would probably include two-stage recarbonation with intermediate settling and lime recovery by recalcination) would have to show an advantage of about 1.9¢/1,000 gal (0.5¢/cu m) to be selected over the system using iron.

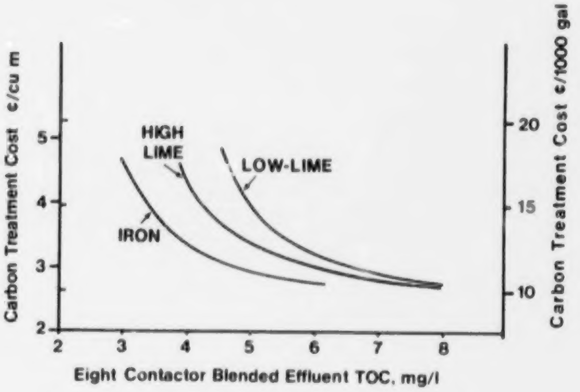


Figure 6. Effect of the three pretreatment systems on the cost of carbon treatment for varying effluent TOC levels.

The results obtained in this study are obviously specific to the type of waste and the conditions of operation discussed here. Though slight differences in performance of the three carbon systems were observed, all three carbon systems produced effluents in essentially the same range of quality. The carbon data analysis illustrates that even slight differences in carbon performance can be related to economics for inclusion in the treatment process selection procedure.

Removal of Heavy Metals by P-C Processes

Phase 1 (2)

Concurrent with the previously described study, heavy metals were added to the influent of each pilot plant. Samples of spiked raw sewage, clarifier effluent, filter effluent, carbon effluent, sludge and filter backwash were analyzed for the specified metal. The analytical determinations were verified by mass balance procedures. Table 4 shows the metal concentrations remaining in the final effluents. The concentration of metal added to each system was 5 mg/l except for mercury, which was 0.5 mg/l. For comparison, the U.S. EPA water quality criteria for metals in potable water sources are also shown.

TABLE 4 - RESIDUAL METALS CONCENTRATIONS FROM THREE SYSTEMS, μg/l

Metal	Quality Criteria	Iron System	Low Lime System	High Lime System
Mn	50	3820	235	37
Ni	-	37	38	12
Zn	5000	192	561	584
Cu	1000	155	169	352
Cd	10	50	44	14
Ba	1000	271	25	942
Pb	50	30	29	19
Cr^{III}	T 50	6	16	18
Cr^{VI}		21	87	76
As	100	58	915	770
Hg	2	-	-	54

Mechanisms of removal include precipitation as hydroxides, hydrous oxides, carbonates and sulfates with subsequent removal by sedimentation and filtration. Activated carbon also removes certain metals by adsorption. These carbon columns were anaerobic, thus providing a suitable environment for the reduction of sulfate to sulfide. The concentration of sulfide was widely variable but precipitation and filtration of the heavy metals as the insoluble sulfide must certainly have occurred in the carbon columns.

Phase 2 (3)

After completion of the first phase of the metals study and the chemical pretreatment study discussed previously, the pilot plant underwent major revision. The flow scheme remained essentially the same, but changes were made in the equipment. The clarifiers were made deeper to allow greater detention time. The 4-in (10 cm) carbon columns were replaced with two 5.75 in (14.6 cm) columns in series. The depth of carbon in each column was 8 ft (2.4 m) for a total carbon depth of 16 ft (4.9 m). The hydraulic loading was 4 gal/min/sq ft (10 m/hr) resulting in 30-min contact time (based on empty bed volume).

During the second phase of the metals study, the raw sewage was spiked with some of the less common metals. The clarifier was operated using either iron (ferric chloride, 40 mg Fe/l), lime (415 mg $Ca(OH)_2$/l, pH 11.5) or alum (220 mg/l). The data are summarized in Tables 5, 6 and 7. It can be seen that most of the metal removal occurs in the chemical clarification step. However, carbon does remove a significant fraction of some of the metals applied. For example, in improving the removal of mercury from 94% to 98.3% (Table 5), the carbon reduced the residual concentration from 3.6 ppb to 1 ppb, 56% removal across the carbon. Table 6 shows that where lime was not particularly effective for mercury, antimony, selenium, thallium, and vanadium, carbon increased the total removal significantly.

This project has now produced a large body of data relating to the fate of many heavy metals in independent physical-chemical treatment processes. The combination of chemical clarification, filtration and activated carbon adsorption is effective for the removal of most heavy metals, assuming the proper choice of chemical. Molybdenum, for example, was not removed by alum or lime, but the ferric chloride-carbon process gave 80% removal. The metal residual after clarification and filtration should be somewhat predictable

TABLE 5 - METALS DATA SUMMARY, ALUM COAGULATION

Metal	Oxidation State	Initial Concentration PPM	Cumulative % Removal			PPB Residual Metal
			Clarifier	Filter	Carbon	Carbon
Silver	+1	0.6	95	96.9	99.2	5
Beryllium	+2	0.1	93	98.1	98.9	1
Bismuth	+2	0.6	92	95.5	96.9	19
Cobalt	+2	0.8	47	49	56	352
Mercury	+2	0.06	88	94	98.3	1
Molybdenum	+6	0.6	4	0	10	540
Antimony	+3	0.6	61	62	71	174
Selenium	+4	0.5	53	48	56	220
Tin	+2	0.6	92	95.3	94	36
Titanium	+4	0.6	95.3	>95.8	>95.8	<25
Thallium	+1	0.6	33	31	39	366
Vanadium	+5	0.5	86	94	>99.5	<3
Manganese	+2	0.7	31	30	33	469
Nickel	+2	0.9	25	25	37	569
Zinc	+2	2.5	1	1	28	1800
Copper	+2	0.7	70	67	98.3	12
Cadmium	+2	0.7	43	45	55.5	312
Barium	+2	0.5	87	79	92	40
Lead	+2	0.6	91	95.5	96.6	20
Chromium	+3	0.7	93	97.6	99.3	5
Chromium	+6	0.7	61	64	97.4	18

based on solubility products and jar tests. Carbon provides additional removal of most metals, the extent of which can best be determined by pilot column operation.

Effects of pH on Removal of Organics from Wastewater by Activated Carbon

Early research on activated carbon treatment of secondary effluent seemed to indicate that adsorption of refractory organics improved with decreasing pH. Later work, however, showed that pH effects were different for different compounds. The manipulation of pH is not uncommon in physical-chemical systems, occurring in lime clarification and neutralization and also incidentally when using the acid salts, ferric chloride or alum. The adjustment of pH is a relatively easy operation. Thus, a P-C carbon system could easily be operated at any reasonable pH if adsorption were improved.

One of the Cincinnati P-C pilot plants was outfitted to provide data on the

TABLE 6-- METALS DATA SUMMARY, IRON COAGULATION

Metal	Oxidation State	Initial Concentration PPM	Cumulative % Removal			PPB Residual Metal
			Clarifier	Filter	Carbon	Carbon
Silver	+1	0.5	94	98.2	99.1	5
Beryllium	+2	0.1	93	94	98.9	1
Bismuth	+2	0.5	83	94	96.2	19
Cobalt	+2	0.5	27	18	30	350
Mercury	+2	0.05	92	98	99	<1
Molybdenum	+6	0.6	66	68	80	120
Antimony	+3	0.5	60	65	72	140
Selenium	+4	0.1	66	75	80	20
Selenium	+4	0.05	68	80	75	13
Tin	+2	0.5	95	98.0	98.5	8
Titanium	+4	0.5	84	87	90	50
Thallium	+1	0.6	36	30	45	330
Vanadium	+5	0.5	93	97.2	97.8	11

TABLE 7-- METALS DATA SUMMARY, LIME COAGULATION

Metal	Oxidation State	Initial Concentration PPM	Cumulative % Removal			PPB Residual Metal
			Clarifier	Filter	Carbon	Carbon
Silver	+1	0.5	95.8	97.1	98	10
Beryllium	+2	0.1	97.8	99.4	99.5	1
Bismuth	+2	0.6	90	95.3	96	24
Cobalt	+2	0.5	90	91	95	25
Mercury	+2	0.5	45	70	91	45
Molybdenum	+6	0.5	NO REMOVAL			-
Antimony	+3	0.6	21	28	52	288
Selenium	+4	0.5	36	35	95	25
Selenium	+4	0.06	46	38	67	20
Tin	+2	0.5	91	92	92	40
Titanium	+4	0.5	92	95.5	95.3	24
Thallium	+1	0.5	54	60	72	140
Vanadium	+5	0.5	55	57	91	45

influence of pH on carbon performance. Raw sewage was clarified with 194 mg alum/l. The dosage was manually adjusted to maintain pH 6.4 ± 0.2. After dual-media filtration at 3.9 gal/min/sq ft (9.5 m/hr), the effluent was split into three streams. The pH's of two of the streams were adjusted to 4.0 and 8.8, respectively, and pumped to separate carbon adsorbers. The third stream was pumped at ambient pH (6.4) to a third adsorber. These were identical adsorbers treating the same wastewater at three different pH's. The adsorbers were 5.75 in (14.6 cm) two-stage series with 8 ft (2.4 m) of Filtrasorb 300 carbon per stage for a total carbon depth of 16 ft (4.9 m). The hydraulic loading was 6 gal/min/sq ft (15 m/hr) resulting in 20 min total empty bed contact time. Sodium hydroxide and sulfuric acid were injected by pH controlled metering pumps into the carbon column feed streams upstream of in-line mixers. There was no difficulty maintaining constant pH.

Table 8 shows the results of the alum clarification-filtration treatment of raw sewage over the six months study. Alum clarification-filtration was highly effective for removal of total organic carbon (TOC), suspended solids (SS), phosphorus (P) and turbidity. Around 40% removal of soluble TOC (STOC), color and total Kjeldahl nitrogen was observed.

Operation of the carbon columns at pH 4.0, 6.4 and 8.8 did produce varying results. Figure 7 shows breakthrough of TOC with throughput. The low pH system performed poorly from the start, and was exhausted rather quickly. The high pH system provided the best effluent through most of the study. There was an apparent alteration of the wastewater caused by elevating the pH to 8.8. This was evidenced by much higher solids concentration in the carbon column backwash water than from the others. This led to the suspicion that precipitation and filtration of soluble or colloidal organics was providing improved removal. However, analysis of data on the ratio of particulate TOC to total TOC showed no significant difference at the three pH's. The performance of the column which received pH 6.4 wastewater was poorer than the high pH column at the beginning, but declined at a slower rate, indicating the development of some biological activity as the study progressed. The pH 6.4 column was the only one which discharged any sulfide (0.5 mg/l), indicating a more suitable environment for biota than the other two. Biological activity was probably not a major factor because of the low feed organics (15 mg TOC/l) and the fact that the beds were kept very clean by daily air-scour, water backwash.

TABLE 8 - POLLUTANT REMOVAL BY ALUM CLARIFICATION AND FILTRATION

Parameter	Raw Sewage	After Clarification and Filtration	% Removal
TOC, mg/l	96.1	15.1	84
STOC, mg/l	23.6	13.5	43
SS, mg/l	235	7.8	97
Color, units	42	25.4	40
P, mg/l	7.3	0.3	96
Alk., mg $CaCO_3$/l	186	101	-
Turbidity, units	84	5.2	94
NH_3, mg N/l	11.2	11.1	-
TKN, mg/l	21.3	12.6	41

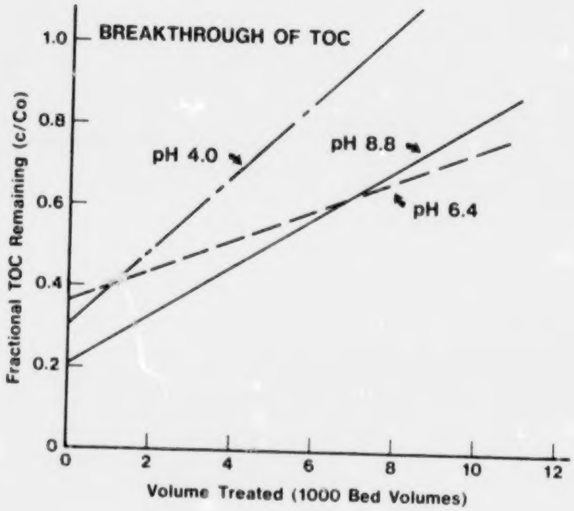


Figure 7. Breakthrough of TOC.

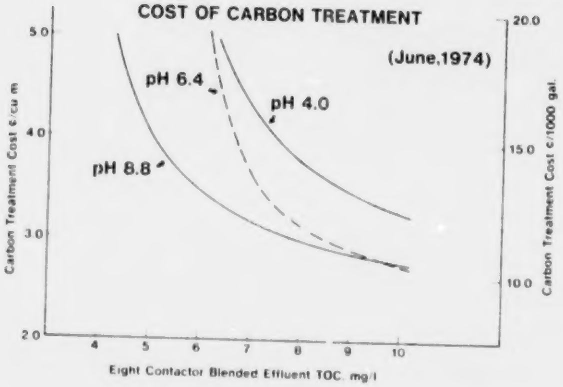


Figure 8. Cost of carbon treatment.

After the six-months study was completed samples of carbon from the first eight feet of each system were subjected to thermogravimetric analysis. Weight loss at 475°C in a nitrogen atmosphere was measured. The pH 6.4 and 4.0 system spent carbon samples showed approximately equal weight loss, while the weight loss of the pH 8.8 spent carbon sample was about 50% higher. While this measurement may not correlate quantitatively with organics adsorbed on the spent carbon, it does indicate that the higher pH was favorable for adsorption of the organic materials in the wastewater studied. It also indicates little difference in adsorption at pH 4.0 and 6.4, implying that the obvious improvement in removal at pH 6.4 over pH 4.0 might be the result of biological activity. This weight loss analysis is very limited data, and the implications suggested here are only speculative.

Figure 8 illustrates the results of a data analysis similar to that outlined earlier for the study on effects of chemical pretreatment. Cost analysis assumptions are the same as those used earlier and as shown in Table 3. This shows total carbon treatment cost as a function of effluent TOC. At low effluent TOC requirements, operation at the higher pH is the obvious choice. At less stringent effluent standards (<50% removal across the carbon), there is little to choose between the pH 6.4 and 8.8 operation. Operation of a carbon column at low pH is not indicated for this wastewater.

Formation and Removal of Chloro-organics in a Tertiary Breakpoint Chlorination-Activated Carbon System

At the request of EPA Region III, EPA Cincinnati is undertaking an extensive pilot study of the formation of chlorinated organics during the process of ammonia removal by breakpoint chlorination (BPC). Region III has awarded a construction grant to the Washington Suburban Sanitary Commission (WSSC) for the construction of a 60 mgd (230,000 cu m/day) advanced waste treatment plant in Montgomery County, Maryland. Region III is in the process of preparing an Environmental Impact Statement for the Montgomery County facility. The question of the extent of formation and removal of chlorinated organics during the breakpoint process was encountered because of recent innovations in analytical chemistry which disclosed the presence of these possible carcinogens in many municipal water supplies. Since the Montgomery County plant effluent will discharge to the Potomac upstream of the drinking water intake of Washington, D.C., Region III asked MERL to supply data to indicate the extent of the chloro-organics problem likely to be encountered.

A brief preliminary study was conducted wherein batches of lime clarified and filtered secondary effluent were pumped through a BPC system followed by a short contact time (2-1/2 min) carbon column. This was done to provide quick data to the Region, recognizing the limitations of poor chlorination control and unrealistic operating parameters. During eight days of operation BPC and short contact carbon treatment produced the results shown in Table 9. Only chloroform was detected above 1 µg/l in the lime clarified and filtered activated sludge effluent. Only small quantities of the haloforms were found after breakpoint with the exception of chloroform and bromodichloromethane. The concentrations for these two compounds are in the same range as values observed by others in the City of Cincinnati tap water (45 µg chloroform/l and 13 µg bromodichloromethane/l). The TOC of the lime clarified-filtered secondary effluent was 10 mg/l.

During the eight days of this study, the carbon was effective in removing the small amounts of halomethanes formed by chlorination, with the exception of chloroform. Figure 9 shows the formation and removal of chloroform. Breakthrough from the carbon column began on about the second day and continued to complete breakthrough on the last day of the run. The chloroform loading at complete breakthrough was 73 µg $CHCl_3$/g carbon.

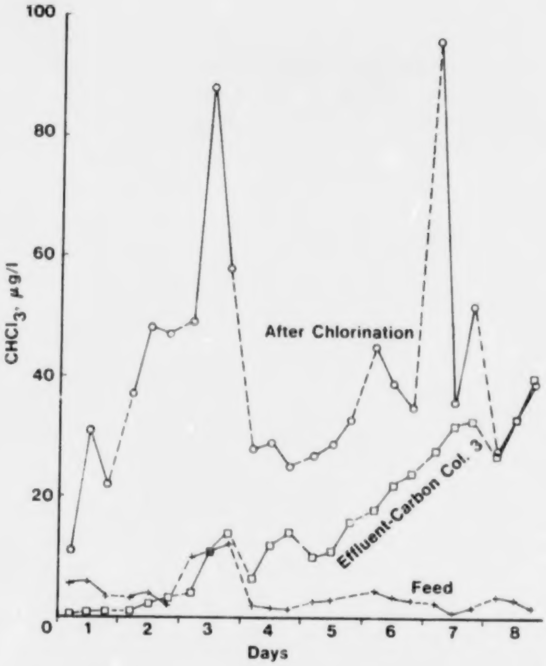


Figure 9. Formation and removal of chloroform.

While the preliminary study discussed above was being carried out, the P-C pilot plant formerly used for the study on effects of pH was being revised for an extensive study of the BPC-carbon system. Trickling filter effluent will be supplied as pilot plant feed. Lime feeding and acid neutralization controlled by automatic pH equipment was installed. Again, three parallel

TABLE 9 - FORMATION AND REMOVAL OF VOLATILE CHLORINATED ORGANICS

	Feed	B.P. Cl_2 Eff.	Carbon Eff.
$CHCl_3$	1-12 (4 avg.)	11-96 (40 avg.)	0.1 ⟶ breakthrough
$C_2H_4Cl_2$ (a)	ND - 0.7	ND - 2.5	ND - 0.2
$BrCHCl_2$	ND - <0.1	<0.1 - 22	ND - 0.3
Br_2CHCl	ND - <0.1	ND - 0.3	ND - <0.1
$CHBr_3$	ND - <0.1	ND - 0.8	ND - <0.1

(a) Single peak for $C_2H_4Cl_2$ and CCl_4 - expressed $C_2H_4Cl_2$

(b) ND - no peak observed

(c) <0.1 - detectable but too small to quantify

(d) All values µg/l

(e) Three grab samples per day

carbon systems will be used to treat filter effluent. One will incorporate a BPC system between the filter and the first stage of carbon. The second will have a BPC system installed between the first and second stages of the carbon system. The third carbon adsorber will not have a BPC system.

The ammonia content of the influent to each BPC system will be monitored by a Technicon Auto Analyzer. The ammonia concentration will be transmitted as an electrical signal to a sodium hypochlorite feed pump controller. This will cause the feed pump to add chlorine at a predetermined ratio to achieve breakpoint. Success of ammonia oxidation will be monitored by analyzing effluent streams after carbon treatment. Free and total chlorine will also be measured by the Auto Analyzer at various points in the system. The pH of the chlorination reactions will be controlled automatically. The carbon systems will be operated upflow with 8 ft (2.4 m) of Filtrasorb 400 per stage for a total depth (at repose) of 16 ft (4.9 m). The hydraulic loading will be 6 gal/min/sq ft (15 m/h) for a total contact time of 20 min (based on empty bed volume, carbon at repose).

As soon as the major process equipment revisions could be completed, one BPC system was operated for a period of three weeks with manual control of the chlorine feed rate. The BPC system that preceded carbon treatment was the one chosen for this run. The operator would take the ammonia reading from the on-line Auto Analyzer and manually set the hypochlorite feed pump to deliver the appropriate chlorine dose. Free and total chlorine were measured amperometrically and the carbon effluent ammonia content was monitored by the in-line Auto Analyzer.

The data from the three-week run have, as of this writing, not yet been completely analyzed. However, certain preliminary observations can be made. The halomethanes were formed at concentrations somewhat higher than observed earlier. The feed to the BPC system varied considerably, from 5 μg/l to 250 μg/l. Figure 10 shows the chloroform concentration in the influent to the carbon system (after BPC), the effluent from the first stage carbon and the effluent from the second stage carbon column. Breakthrough of chloroform through the first 8 ft (2.4 m) of carbon began very quickly. However, the final effluent never exceeded 10 μg/l. This could indicate that contact times longer than 10 min are required for efficient chloroform removal. This question will be addressed in more detail in future work.

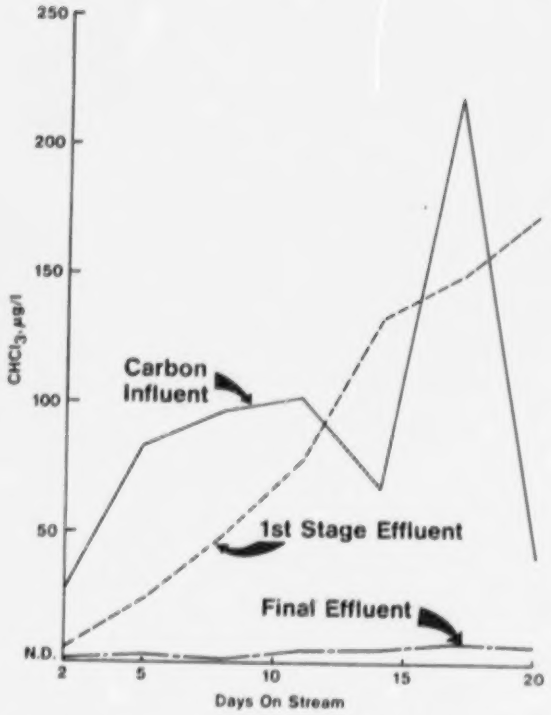


Figure 10. Chloroform removal by activated carbon.

The data showing that effluent from the first stage carbon at times was higher than the influent implies that the adsorption of chloroform is easily reversible. This will also be investigated further.

When all the automatic control equipment is installed, the three carbon systems will go on-stream continuously in order to provide data on not only the formation and removal of halomethanes, but also on the oxidation of ammonia with chlorine, the effect of chlorine on carbon life, and control and operational problems involved with the BPC system. Questions on the halomethane problem that will be answered include:

1. Correlation between chlorine dose or residual and halomethane production.

2. Effect of precursor removal on halomethane production (BPC after first stage carbon).

3. Capacity of carbon to remove halomethanes.

4. Influence of carbon contact time on removal of halomethanes.

Enhancement of Granular Activated Carbon Performance by the Addition of Nitrate

Under contract with EPA, the Los Angeles County Sanitation District constructed and operated a 50 gal/min (3.2 l/sec) P-C pilot plant at Pomona, California (4). The system consisted of chemical clarification of raw sewage using alum and polymer followed by treatment in a downflow carbon column. Performance and system parameters are shown in Table 10. The major objective of the project was to learn what problems could result from long term treatment of chemically clarified raw sewage. It was realized that biological growths in the P-C carbon systems might cause severe problems of plugging and/or sulfide generation. Thus, the Pomona project was designed to quantify these effects and to investigate measures to alleviate the problem conditions, should

TABLE 10 - POMONA P-C-T PILOT PLANT

Parameter	Raw Sewage	Clarifier Effl.	Carbon Effl.
TCOD, mg/l	321	96	19
DCOD, mg/l	49	49	14
SS, mg/l	199	28	7
Turb., JTU		23	6
BOD_5, mg/l		36	8
Color units		20	8
P, mg/l	11.1	1.3	0.9
NO_3, mg N/l		0.9	1.3
pH	7.7	6.8	6.8

Clarifier

Flow - 60 gal/min (3.8 l/sec) constant
Overflow Rate - 1180 gal/day/sq ft (48 cu m/day/sq m)
Weir Loading - 10 gal/min/ft (2 l/sec/m)
Detention Time - 84 min
Alum Dose - 25 mg Al/l
Polymer Dose - 0.3 mg/l - anionic
Sludge Production - 2000 lb/mil gal (250 mg/l)

Granular Carbon

Flow - 50 gal/min (3.2 l/sec) constant
Hydraulic Loading - 4 gal/min/sq ft (10 cu m/hr/sq m)
Contactor Type - Single Stage Packed Bed Downflow
Carbon Size - 8x30 mesh
Carbon Depth - 16 ft (4.9 m)
Empty Bed Contact Time - 30 min
Sodium Nitrate Dose - 33 mg $NaNO_3$/l
1st Cycle Carbon Loading - 3.5 g TCOD removed/g carbon (1.5 g DCOD/g)
1st Cycle Carbon Utilization rate - < 173 lb/mil gal (21 mg/l)

Note: Carbon column ran 18 months (1st cycle) prior to regeneration and was not exhausted. Three regeneration runs were conducted in order to obtain regeneration data. Performance data shown above is average data over entire 27-month operation.

they appear. Of course, process performance would also be monitored. Another important objective was to obtain data on the regeneration of P-C carbon, which would likely be heavily loaded with organics and possibly carry biological slimes.

As expected, hydrogen sulfide appeared in the carbon effluent after several weeks on stream. Table 11 shows the measures used in attempting to eliminate the sulfide, including backwash methods, aeration or chlorination of carbon influent and nitrate addition. The final solution was the addition of 33 mg $NaNO_3$/1. Nitrate is reduced to nitrogen gas and the resulting oxygen is used as hydrogen acceptor in the biochemical utilization of substrate. Nitrate addition caused additional pressure drop, but it was within the design pressure capacity of the system. The increase in headloss during a 24-hour run between backwashes was always less than [illegible] psi (340 k N/sq m) and on the average less than 30 psi (210 k N/sq m). A welcome result of the nitrate addition at Pomona

TABLE 11 - PERFORMANCE OF THE VARIOUS H_2S CONTROL
AT POMONA

H_2S Control Methods	Carbon Effluent Total Sulfide, Conc., mg/1 S	
	Average	Range
(1) Surface wash - Backwashing Technique	2.86	1.0 - 5.7
(2) No. (1) Plus intermittent O_2 addition to carbon column at D.O. level ≈ 4 mg/1	1.85	1.4 - 2.5
(3) Surface wash + Air/water backwash plus oxygenation of the chemically clarified effluent to D.O. 2-6 mg/1	1.87	0.8 - 3.0
(4) No. (3) + 20 mg/1 Cl_2[a] added to carbon influent	1.74	0 - 4.3
(5) No. (3) + 40 mg/1 Cl_2 added to carbon influent	1.13	.09 - 2.6
(6) No. (3) + 2.9 mg/1 N [b] added to carbon influent	0.3	0 - .95
(7) No. (3) + 4.5 mg/1 N	0.13	0 - 0.60
(8) No. (3) + 5.1 mg/1 N	0.05	0 - .26
(9) No. (1) + 5.3 mg/1 N	0.019	0 - .10
(10) No. (1) + 5.4 mg/1 N	0	0 - .05

[a] added as sodium hypochlorite solution

[b] added as sodium nitrate solution

and one not entirely expected was an apparent indefinite extension of the life (capacity) of the carbon. The carbon column was operated for 18 months without replacement and without a significant decline in organic removal efficiency. The apparent carbon loading during that time reached a phenomenal 3.5 g COD removed/g carbon of which 1.54 g/g was soluble COD. Considering that values in the range of 0.5 g COD removed/g carbon are commonly accepted for carbon loading in physical-chemical systems, it is apparent that the mechanisms of filtration (particulate COD "loading" was 3.5-1.5 = 2 g/g) and nitrate assisted biological activity contributed greatly to the organics removal.

In order that the Pomona experience could be verified and expanded at another location, the Lebanon pilot plant facility is currently modifying an existing P-C pilot system to run an upflow carbon column on lime clarified-neutralized raw wastewater. Sodium nitrate will be added prior to the carbon column, and the performance will be monitored for a long period of time. It may be possible that nitrate oxygen will enhance the biological activity within a carbon column such that the replacement rate of the carbon is significantly reduced or even eliminated.

FULL SCALE TREATMENT OF MUNICIPAL WASTEWATERS BY ACTIVATED CARBON

There are several tertiary activated carbon systems now in operation producing effluents of reuse quality. South Tahoe effluent is pumped to a reservoir which is used for sport fishing and as an irrigation water supply. Colorado Springs carbon effluent is used as power plant cooling water. Orange County Water District, with a system similar to Tahoe's, will inject the carbon effluent, after blending with low TDS water, into the underground aquifer to prevent salt water intrusion. The use of reclaimed wastewater (including carbon treatment) for augmentation of drinking water supplies is being seriously considered in certain water-short areas of this country. Direct potable reuse has been routinely practiced at Windhoek, South Africa.

The first full-scale facility in the United States which treats a waste of primarily domestic origin by only P-C processes is now on stream at Rocky River, Ohio. This 10 mgd (38,000 cu m/day) facility utilizes alum and polymer to clarify raw sewage in existing primary clarifiers which have been equipped with internal flocculation mechanisms. Clarifier effluent is then pumped through a bank of eight parallel carbon adsorbers. The carbon effluent is then discharged to Lake Erie. The Research and Development Program of EPA and its predecessor organizations has helped to finance the construction of the activated carbon facility via an R&D grant. Following the shakedown and upon initiation of full flow through the carbon system, a one-year evaluation period will begin. Complete records of operating conditions and performance will provide essential information on the effectiveness of the P-C system at full scale. Extensive cost and time accounting will allow determination of the actual cost of the system and all subsystems. The man-hour requirements for operational duties, preventative maintenance and unscheduled maintenance will be catalogued so that the data will be relevant not only to this combination of processes but to any other system containing one or more of the processes used at Rocky River.

After the one-year evaluation period the performance, operating and cost data will be compiled in a technical report. At that time, the U.S. consulting engineers and wastewater treatment agencies will have access to the experience and data gathered at the first full-scale P-C facility. This will be a major addition to the body of knowledge in the municipal wastewater treatment field.

SUMMARY

The removal of organic materials from municipal wastewater by activated carbon has been practiced for several years in the tertiary systems, and now is being demonstrated in a P-C system. Much knowledge has been gained over the last decade, and carbon system design is no longer a mystery. However, there is still much to learn about the mechanisms of removal of exotic and commonplace organic compounds by this versatile adsorbent. The factors affecting adsorption of specific compounds or classes of compounds have not been fully defined nor have methods of prediction of adsorption performance.

As technology develops and more and varied organic materials enter and are detected in the water environment, activated carbon can be expected to assume an expanding role in environmental protection.

References

1. Westrick, J.J., and Cohen, J.M., "Comparative Effects of Chemical Pretreatment on Carbon Adsorption." Presented at the WPCF 45th Annual Conference, Atlanta, Georgia, (Oct. 1972). In Press.

2. Maruyama, T., Hannah, S.A., and Cohen, J.M., "Metal Removal by Physical and Chemical Treatment Processes," Jour WPCF, 47, 962 (May 1975).

3. Hannah, S. A., Jelus, M., and Cohen, J.M., "Removal of Uncommon Trace Metals by Physical and Chemical Treatment Processes." Presented at the 48th Annual Conference, WPCF, Miami Beach, Florida (Oct. 1975).

4. Directo, L.S., Chen, C.L., and Kugelman, I.J., "Pilot Plant Study of Physical-Chemical Treatment." Presented at the 47th Annual Conference, WPCF, Denver, Colorado, (Oct. 1974).

NEW INDUSTRIAL WASTEWATER SEPARATION PROCESSES DEVELOPED UNDER THE EPA RESEARCH PROGRAM

J. Ciancia
Industrial Environmental Research Laboratory-Cincinnati
Edison, New Jersey 08817

ABSTRACT

This paper describes the Industrial Research and Development Program of the U. S. Environmental Protection Agency followed by a brief discussion of an approach companies should use for abating pollution at manufacturing plants. Separation technologies involving chemical recovery and/or wastewater reuse that have been developed under the Industrial R&D Program of EPA are then discussed. The technology developments covered are (1) a novel ion exchange technique for separating acids from metals to permit the reuse of spent processing baths, (2) an ion exchange process for recovering chromate from pigment manufacturing wastewaters, (3) the development and in-plant demonstration of reverse osmosis and electrodialysis for recovering chemicals and reusing water from rinse waste discharges in the metal finishing and fabricating industries, (4) a novel approach for increasing the heat and mass transfer while also preventing scaling in vertical tube evaporators and its demonstration on a pilot plant scale for several inorganic wastewaters, and (5) a closed-loop type treatment approach involving electrolytic copper recovery, process changes and integrated chemical treatment of rinse waters that have been applied on a full scale for abating pollution at a copper and brass wire mill.

INTRODUCTION

The significance and complexity of the industrial pollution problem coupled with recent governmental legislation has spurred the development of new and improved technology for treating and/or recovering the multitude of wastes that spew from the Nation's industrial plants.

With the passage of the Clean Water Restoration Act of 1966, the U. S. Environmental Protection Agency assumed a key role in the development of more effective and economical technology for treating industrial wastes. Under the provisions of this Act, EPA initiated a program to develop and demonstrate new and improved technology for the prevention, control, treatment, recovery and reuse of industrial wastes. The Agency's authority to develop improved industrial and joint industrial/municipal wastewater treatment systems was expanded with the passage of the Federal Water Pollution Control Act of 1972. Moreover, this legislation establishes closed-loop type technology as the direction to pursue in developing new pollution abatement approaches by declaring that it is the national goal to eliminate the discharge of pollutants into the navigable waters of the United States by 1985. As a result, the EPA Research, Development and Demonstration Program has become more strongly oriented toward technology that can accomplish chemical recovery and water reuse at the same facility or through by-product recovery of the waste materials.

The Industrial Research and Development Program in EPA is carried out by inhouse efforts, grants and contracts. However, the bulk of the studies to date have been carried out through the grant and contract mechanism, which permits the program to utilize the best science and engineering talent in the Nation's universities, private research institutions, and industry.

The objectives of the program are to assess the pollution problems and current abatement practices in relation to the needs of the industries, and support research, development and demonstration studies involving technology with a high potential for solving the most significant problems at the least cost and with a maximum amount of chemical recovery and water reuse. To warrant support, it is necessary that preliminary investigations show that the proposed technology is technically feasible and more attractive than existing commercial approaches for abating specific waste problems. Studies are supported at all stages of development from the bench scale through pilot plant and full scale demonstrations.

ABATEMENT OF INDUSTRIAL WASTE PROBLEMS

In approaching pollution control, each plant must first define the problem by identifying all waste sources, including the characteristics, volumes and pollutant loads associated with each discharge. With the problem defined, process change and in-plant control techniques can now be considered and implemented where feasible for conserving water and eliminating all unnecessary wastes.

Since it is much simpler to recover components before waste streams are combined, and thus become complex mixtures of contaminants, the next step in a pollution control program is to carefully assess the different waste sources to explore the application of closed-loop type technology. Where chemical recovery and water reuse do not appear to be feasible, the application of alternative treatment technology should then be considered for the remaining waste streams on an individual and/or combined basis to meet governmental effluent requirements with a minimum of capital, operating and overall environmental costs.

NEW ABATEMENT TECHNOLOGY

The purpose of the remainder of this paper is to provide some insight into closed-loop type developments associated with EPA's Industrial R&D Program and the application of these approaches to industrial pollution problems for achieving chemical recovery and/or water reuse.

Ion Exchange

Unlike the widespread application achieved in water treatment, the commercial use of ion exchange for processing and waste treatment has been rather limited. However, there has been considerable interest in the use of ion exchange as a pollution control tool in the last several years. This stems from a combination of factors, including the intensified interest in pollution control, the capability of the technique to recover chemicals and purify wastewater, and the availability of improved ion exchange resins.

Recently, a novel ion exchange technique for separating strong acids from metal contaminants was shown to be economically feasible for reclaiming phosphoric acid used in the bright finishing of aluminum. Referred to as "acid retardation," the technique permits the separation of strong acids from their salts by passage of the solution through the corresponding salt form of a strong base ion exchange resin. While the mechanism of the process is not completely understood, acid retardation separations involve the preferential absorption of strong acids on the resin which "retards" or slows down the movement of these acids through the bed relative to the salt. The acid molecule is desorbed from the resin by water, thus eliminating the need for chemicals to regenerate the ion exchanger as in conventional ion exchange treatment processes.

The acid retardation process has been optimized in pilot plant equipment using actual phosphoric acid wastewater from aluminum bright finishing production operations. The study utilized a continuous countercurrent contactor to obtain maximum efficiency in separating the phosphoric acid from aluminum.

The pilot plant study showed that over 75% of the acid content could be recovered from feed streams containing 25-35% phosphoric acid which is generally obtained in the countercurrent rinsing of aluminum parts after bright finishing. In this application, the ion exchange resin accomplishes the separation by absorbing the phosphoric acid while the aluminum remains in the wastewaters and passes out the column in the effluent. The acid is eluted from the bed with water, concentrated by evaporation, and returned to the bright finishing processing operation.

The economic evaluation of the process based on the pilot plant results indicates that the required capital investment could be recovered in about 1.5 years when both the cost of the recovered acid and alternative chemical waste treatment are considered. For those plants where the 35% waste acid can be returned to the supplier, the additional savings afforded by the "acid retardation" system is estimated to return the investment in approximately two years.

In another development, a new ion exchange approach has evolved for recovering chromate and zinc from wastewaters discharged from the manufacture of zinc yellow pigment. A full scale in-plant demonstration showed that the new waste abatement system is highly effective and much more economical than chemical treatment. In general, the technology for treating wastewaters from the inorganic pigment industry is presently limited to pH control, precipitation, and removal of suspended solids by conventional liquid/solid separation techniques.

The new approach involves passing the mother liquor and wash waters through a strongly basic anion exchange resin, which reduced the chromate content of the wastewater from 2,700 ppm to about 1-2 ppm in the in-plant demonstration. The effluent from the ion exchange column is neutralized with sodium carbonate to precipitate the zinc. The insoluble zinc carbonate is removed by filtration and then dried for sale. Regeneration of the ion exchange column is accomplished with a heated alkaline salt to remove the chromate which is reused in the manufacturing operation.

When considering the costs associated with conventional chemical treatment (including sludge disposal) and the savings from the recovered chromate and zinc, the company estimated that only 2-3 years would be required to amortize the capital expenditure of $125,000 for the ion exchange system. It was also concluded from the study that the new ion exchange approach has potential for economically treating other chromate pigment manufacturing wastewaters.

Membrane Technology

The U.S. Environmental Protection Agency has an extensive extramural program underway to develop and demonstrate new membrane technology for abating pollution from metal finishing and nonferrous metal fabricating plants. Techniques such as reverse osmosis and electrodialysis are attractive approaches for treating rinse wastes. When used on individual rinse waters, this type of technology can accomplish simultaneous concentration of the chemicals for return to the processing bath while purifying the wastewater for reuse in the rinsing operation.

The reverse osmosis studies may be grouped into three categories. The first, which has been completed, involved testing all commercially available membranes and configurations on the various major types of rinse waters. In another phase of the program, new membranes with improved properties are being tested, especially those which have the potential for expanding the application of RO to higher and lower pH conditions as well as oxidizing contaminants. The remainder of the program involves carrying out full scale demonstrations under actual plant conditions where attractive applications have been identified in order to firmly establish the economic feasibility of the approach for the particular rinse waste.

At the present time, cellulose acetate in several configurations and hollow fiber polyamide are the only reverse osmosis membranes that are commercially available for use in the treatment of industrial wastewaters. In the pilot plant study, extensive tests were carried out on the use of cellulose acetate tubular and spiral wound and hollow fiber polyamide membranes for treating the major metal finishing rinse waters. Reverse osmosis was found to be generally applicable except for oxidizing conditions such as are encountered with chromic acid, and high and low pH rinse waters. The study showed that the hollow fiber polyamide membrane is suitable over a pH range of 4 to 11 and that the cellulose acetate spiral wound and tubular membranes have reasonably good operating lives from pH 2.5 to 7.

Based on the pilot plant investigation and subsequent demonstrations under actual plant conditions, reverse osmosis has been shown to be a feasible treatment/recovery technique for metal finishing rinse wastewaters. However, the field demonstrations also underscore the need for some preliminary investigation prior to deciding on the use of reverse osmosis because of the complex nature of the processing baths which generally contain proprietary additives.

The field demonstrations were carried out on copper cyanide rinse wastes at two plants and nickel plating rinse waste at one location using a hollow fiber polyamide membrane system. Although no problems were encountered in the pilot plant studies, which used actual plating baths that were diluted to rinse water concentration, the membrane significantly declined in performance at one of the two plants where the system was used to treat copper cyanide rinse wastewater. Subsequent laboratory investigations indicated that the decline in performance was due to some extent to the degradation of the spacer (used for flow distribution within the module) by the brightener additive to the bath. At the other location, however, the flux and rejection achieved by the membrane were relatively stable and the system performed satisfactorily over a sufficient time to conclude that reverse osmosis would be a viable treatment approach for copper cyanide rinse wastewaters. The field test on nickel plating rinse wastes established the feasibility of using hollow fiber polyamide membranes for treating this type of rinse water. Moreover, there have been a number of commercial installations of cellulose acetate reverse osmosis systems for the treatment/recovery of nickel plating rinse wastewaters over the past several years, but the success of this application has never been fully documented.

The application of reverse osmosis for closing the loop on metal finishing rinse wastes is shown on Figure 1. Rinse water from the first tank, which would otherwise be discharged to the drain, is separated by the reverse osmosis system into a "concentrate" and "permeate" stream. The concentrate stream is returned to the processing bath, and the purified or permeate stream is recycled to the rinsing operation. A small evaporator is required for those metal finishing operations where insufficient natural evaporation occurs in the bath.

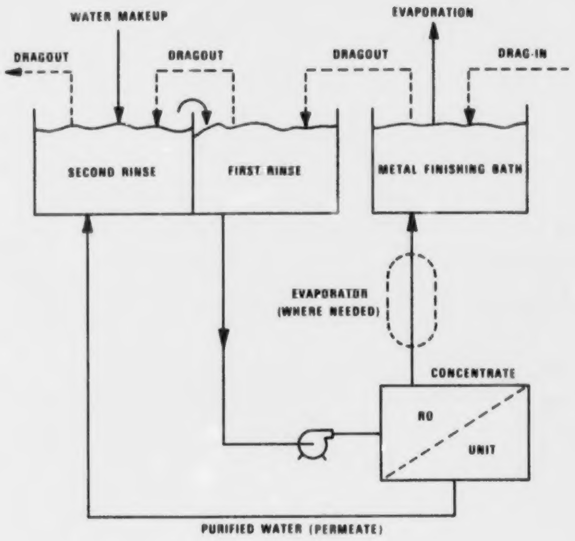


Figure 1
REVERSE OSMOSIS RECOVERY SYSTEM

The major features of reverse osmosis as a treatment device for metal finishing rinse waters are low capital and energy costs, simplicity of operation, and compact equipment requiring a minimum of space. The modular nature of these units makes them particularly attractive for small scale installations.

Electrodialysis is an established water treatment and processing technique which is now starting to receive recognition in the industrial waste treatment field.

Based on a number of pilot plant investigations, the system has been shown to be attractive for purifying metal finishing rinse wastewaters for reuse in the rinsing operations while concentrating the chemicals for return to the processing bath.

In a pilot plant study supported by EPA, a prototype electrodialysis system was used to demonstrate the feasibility of treating and recovering the chemicals from a copper cyanide rinse wastewater. Figure 2 shows the application of the electrodialysis system for achieving closed-loop control of metal finishing rinse wastewaters.

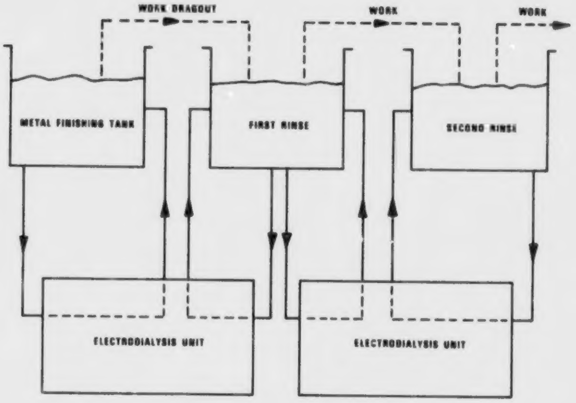

WORK DRAGOUT
WORK
WORK
METAL FINISHING TANK
FIRST RINSE
SECOND RINSE
ELECTRODIALYSIS UNIT
ELECTRODIALYSIS UNIT

Figure 2
ELECTRODIALYSIS RECOVERY SYSTEM

The study showed that the chemicals in the rinse water could be concentrated to over 70% of bath strength. For copper cyanide plating, this level of concentration is sufficient to return all of the chemicals to the processing operation since the bath is operated hot and a significant amount of natural evaporation occurs. The ability of electrodialysis to achieve high levels of chemical concentrations should permit the process to economically close the loop on a number of rinse wastewaters without the need for additional evaporation.

There have recently been a few full scale installations of electrodialysis in metal finishing plants but no data are available at this time for evaluating the feasibility of the technique for achieving closed-loop type control under actual production conditions.

Evaporation

The use of evaporation for treating industrial wastewaters to accomplish water reuse and dissolved solids concentration for simplified disposal or chemical recovery is increasing throughout industry in the United States. The evaporative technique has particular application in the inorganic chemical industry where many wastes cannot be effectively and/or economically treated by the commonly used waste abatement methods. Moreover, in many cases, closed-loop type techniques such as reverse osmosis, electrodialysis and ion exchange cannot achieve the desired concentration and require the use of an evaporator.

Based on the success achieved in the desalting of seawater, a pilot plant investigation has been carried out on a new evaporative technique for industrial wastewater treatment. The approach involves the use of a small amount of surfactant to enhance the heat and mass transfer of vertical tube evaporators while also preventing scaling, fouling and corrosion of the heat transfer surface. The addition of surfactant to the feed achieves enhanced evaporative heat transfer by maintaining a thinned and agitated liquid film on the heat transfer surface, which also results in increased mass transfer by extending the liquid to vapor surface as shown on Figure 3.

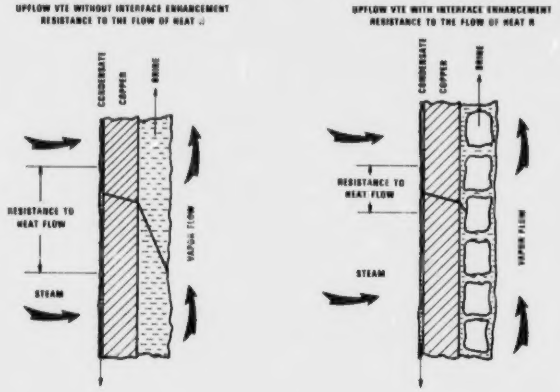

UPFLOW VTE WITHOUT INTERFACE ENHANCEMENT
RESISTANCE TO THE FLOW OF HEAT
CONDENSATE
COPPER
BRINE
RESISTANCE TO HEAT FLOW
STEAM
VAPOR FLOW
UPFLOW VTE WITH INTERFACE ENHANCEMENT
RESISTANCE TO THE FLOW OF HEAT R
CONDENSATE
COPPER
BRINE
RESISTANCE TO HEAT FLOW
STEAM
VAPOR FLOW

Figure 3
CONVENTIONAL versus INTERFACE ENHANCEMENT EVAPORATION

The study involved the use of a 5,000 gal/day pilot plant having double fluted aluminum brass distillation tubes for treating power plant cooling tower and boiler blowdown. The use of surfactant to achieve upflow interface-enhanced Vertical Tube Evaporation provided approximately a 120% increase in heat transfer compared to conventional operation. Another important benefit of the surfactant addition was its ability to inhibit the crystallization of solutes, which permitted the concentration of the wastewaters to smaller volumes.

The results of this study indicate that the new approach to evaporation has considerable potential for the treatment of industrial wastewaters, especially where scaling may be a problem. With significantly increased heat transfer, the size of new evaporators can be greatly reduced or existing systems can be utilized at much higher capacities.

Electrolytic Recovery and Process Changes

A significant pollution problem at copper and brass mills is the disposal of pickling baths when the processing solution becomes ineffective due to the buildup of metallic impurities. In addition, large volumes of dilute rinse waters containing acid, metals and other contaminants are produced in the washing of the work following the pickling operations.

A new approach to the treatment of copper and brass mill wastes involves chemical recovery and water reuse rather than neutralization and precipitation of the pollutants which requires considerable space and results in a potentially troublesome sludge disposal problem. The effectiveness and economics of the new technology has been demonstrated on full scale at a plant producing 75 tons of copper and cuprous alloy per day which used a hot sulfuric acid primary pickle followed by an ammonium bifluoride secondary pickle.

The novel treatment approach, which is shown on Figure 4, involved (1) the installation of an electrolytic copper recovery system in the primary bath to recover the copper and continuously purify the sulfuric acid solution, (2) the replacement of the chromic acid - ammonium bifluoride secondary pickle with a hydrogen peroxide - sulfuric acid secondary pickle containing proprietary additives, and (3) the incorporation of an integrated chemical rinse system to wash the wire prior to fresh water rinsing. The hydrogen peroxide oxidizing treatment in the secondary pickling bath results in the formation of water and a cupric sulfate buildup which is periodically removed by simple crystallization and added to the electrolytic recovery system or sold separately as a by-product. The integrated chemical rinse is a recycle system in which the copper in the pickling solution washed from the wire is precipitated by the alkalinity in the rinse water and settled as a salable dense sludge in a reservoir tank.

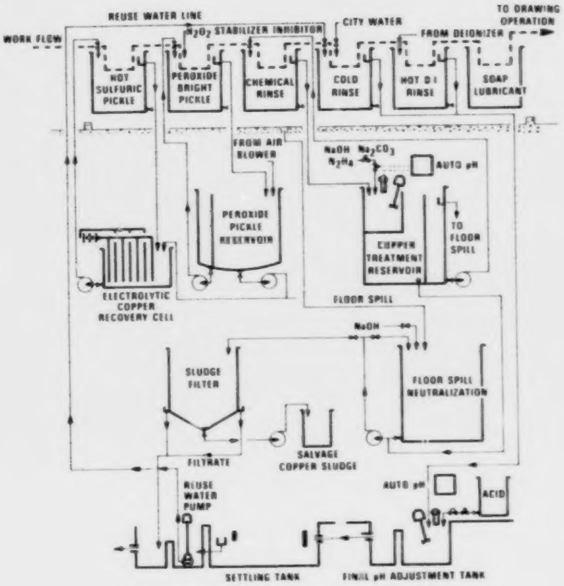

REUSE WATER LINE
WORK FLOW
H_2O_2 STABILIZER INHIBITOR
CITY WATER
TO DRAWING OPERATION
FROM DEIONIZER
HOT SULFURIC PICKLE
PEROXIDE BRIGHT PICKLE
CHEMICAL RINSE
COLD RINSE
HOT D I RINSE
SOAP LUBRICANT
FROM AIR BLOWER
NaOH Na_2CO_3 N_2H_4
AUTO pH
PEROXIDE PICKLE RESERVOIR
COPPER TREATMENT RESERVOIR
TO FLOOR SPILL
ELECTROLYTIC COPPER RECOVERY CELL
FLOOR SPILL
NaOH
SLUDGE FILTER
FLOOR SPILL NEUTRALIZATION
FILTRATE
SALVAGE COPPER SLUDGE
REUSE WATER PUMP
AUTO pH
ACID
SETTLING TANK
FINAL pH ADJUSTMENT TANK

Figure 4

PICKLING OPERATIONS, COPPER RECOVERY, WASTE TREATMENT & WATER REUSE SYSTEM

The application of the recovery approach has demonstrated a reduction of metal losses from 600-700 to less than one pound per day and water consumption from 200,000 to 20,000 gallons per day by chemical rinsing and water reuse. Therefore, the new pollution abatement system has eliminated the dumping of spent pickling baths and the discharge of such troublesome contaminants as chromium, ammonium and fluoride, while permitting the recovery of essentially all of the copper as well as reuse of about 90% wastewater.

The company estimates that when the cost of pickling and waste treatment for the new approach is compared with the pickling and waste costs of conventional chemical treatment, a substantial savings has been achieved -- a daily cost of$194 compared with $540 for conventional abatement.

SUMMARY

In summary, new wastewater treatment technology is emerging for chemical recovery and water reuse but the extent of developments is relatively slow in comparison to the overall needs of the Nation, primarily because of the complexity of industrial pollution problems. By continuing to support research, development and demonstrations of innovative technology, as described in this paper, the U. S. Environmental Protection Agency intends to play a significant role in the effort of attaining the national goal of zero discharge.

STATUS OF INSTRUMENTATION AND AUTOMATION FOR CONTROL OF WASTEWATER TREATMENT PLANTS

J. F. Roesler
MUNICIPAL ENVIRONMENTAL RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

ABSTRACT

This paper addresses the questions about the effectiveness of instrumentation and automation that is being used in wastewater treatment. A user survey of the instruments and automatic controls being used in 50 wastewater treatment plants is described. The instruments were evaluated on the basis of their usage and reliability. Automatic control strategies were evaluated on the basis of their performance improvement and cost effectiveness. Specific examples of control loops are described as implemented at Renton, Washington; Palo Alto, California; and the U. S. Environmental Protection Agency pilot plant at Blue Plains Washington, D.C., automation for both biological and physical-chemical treatment is discussed.

INTRODUCTION

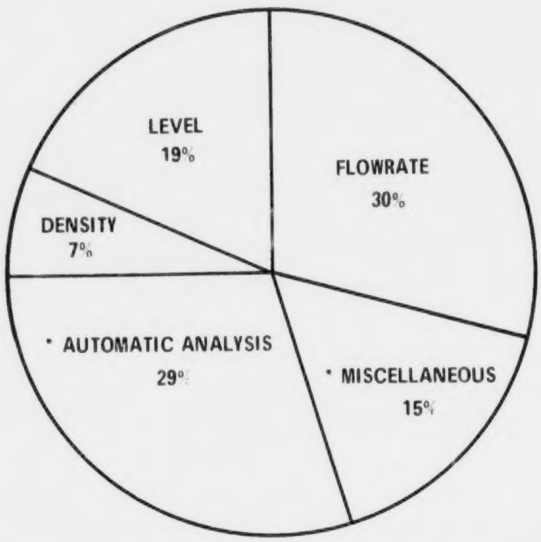
The objectives of the instrumentation and automation of any process are to improve the reliability of maintaining a consistent product quality, enhance process performance and reduce costs. The concept of applying instrumentation and automation to wastewater treatment is still relatively new, and as a result many questions must be answered. Some of these questions are: 1) what equipment is currently being utilized, 2) how effective is automated control and 3) what are the problems that must be overcome to promote automation? Once the answers to these questions are known, productive utilization of instrumentation and automation must be promoted.

In this paper an attempt will be made to answer these questions by describing some of the U. S. Environmental Protection Agency (EPA) research results. This research involves an EPA sponsored user survey of 50 wastewater treatment plants, the evaluation of several control strategies at Palo Alto, California, inhouse efforts evaluating dissolved oxygen (DO) control at Renton, Washington and EPA pilot plant work on physical-chemical treatment in Washington, D.C. To maintain communication with the user community the establishment of an EPA advisory committee is also described.

CURRENT STATUS

In order to answer the first question, a user survey of 50 wastewater treatment plants (1) was conducted. Each plant was personally visited by the interviewer who determined the type and extent of usage of instruments and automatic controls. In each plant the operational personnel were asked to give an estimate as to the effectiveness of the sensor or control loop. The degree to which each instrument was used in each plant and the operator's responses were used to determine the effectiveness of the equipment.

The present use of specific types of sensors was evaluated by considering the distribution of all types of sensors in all 50 plants as shown in Figure 1. Every plant had some device for monitoring flow. Thus 30% of all the instruments in all of the plants were for flow measurement (Figure 1). Automatic analyzers were the next highest category probably because this category included, turbidity, conductivity, pH, DO, chlorine and other analyzers. The miscellaneous analyzers section that is indicated in Figure 1 include such analyzers as rotational speed, weight, and position, etc.



* NON-LABORATORY PROCESS INSTRUMENTS ONLY

Figure 1. Observed distribution of measuring instruments

In Figure 2, the instruments observed during the survey are summarized according to the criteria of unsatisfactory (abandoned equipment), fair (performance considered marginal or excessive maintenance is required) and satisfactory. Note that with the exception of such devices as the bubbler-type level detectors, venturis, temperature guages, etc., most instruments suffered a 31% less than fair performance record. Furthermore, instruments manufactured by the same manufacturer and of identical model were abandoned at some locations but were utilized at other locations. Note also that complicated equipment, such as the TOC analyzers grouped in the "other analytical analyzers" section, suffered very high failure rates. Simple equipment such as bubbler-type level detectors performed well and were well integrated into automatic control systems as shown in Figure 3. Here liquid level is the most popular of all observed control schemes. With the exception of computers, as the control scheme becomes more complicated the number of poor experiences increase.

AUTOMATION OF ACTIVATED SLUDGE PROCESSES

To answer the second question as to the effectiveness of automation, more detailed studies are necessary. The technique usually suggested for such an evaluation is the comparison of the performance of the plant under automatic control with that of manual operation. However, the standards for manual operation vary according to the idiosyncrasy of each plant and of each operator. It is therefore, necessary that the manual operation be well defined and rigidly enforced. Two studies that meet these requirements were carried out at Renton, Washington (2) and Palo Alto, California (3).

The Renton plant was operated for about a year (March 1970 to April 1971) under manual control while an automatic DO control system was being installed into a new aerator. The following year, the plant was successfully operated with automatic DO control. Data were collected for comparative purposes during the months of October, November and December for years 1970 and 1971. The operators and plant management had an excellent attitude toward automation. Also, the manual control policy was well defined and expertly carried out.

The obvious question, however, is whether the sewage was identical for both time frames. One partial answer is that the BOD loading to the plant increased about 50% during the automatic control

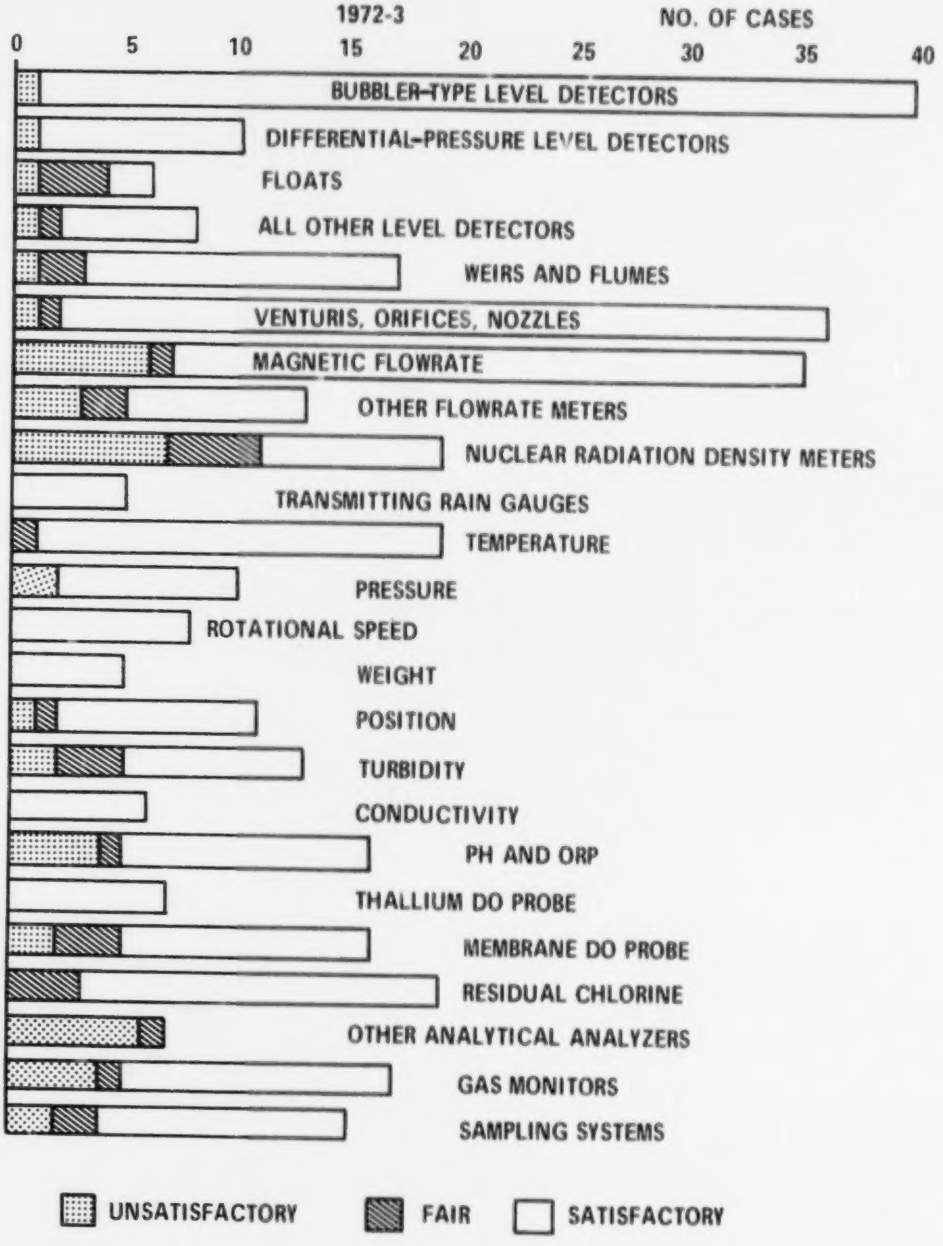


Figure 2. Performance summary of measuring devices in wastewater- treatment facilities

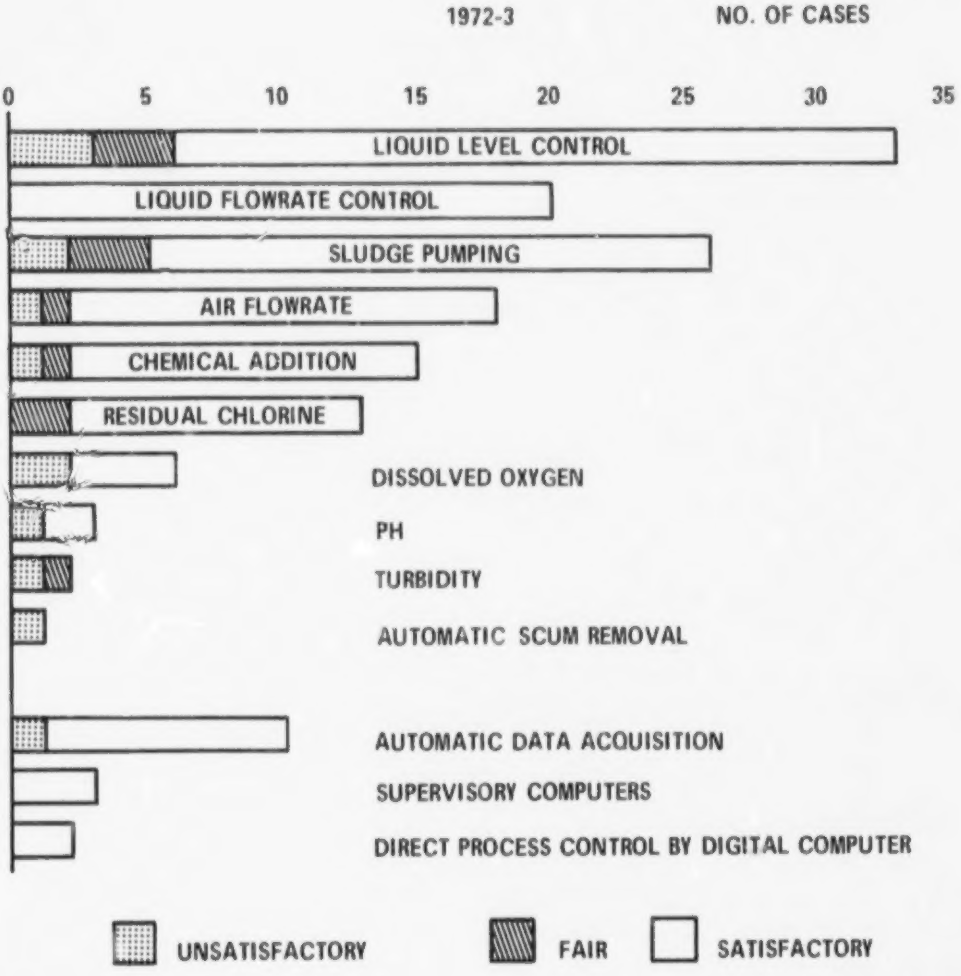


Figure 3. Summary of automatic control experiences in wastewater- treatment facilities

period. In spite of this increase, the performance of the plant did improve. The effluent BOD decreased from a geometric mean of 11.1 ppm, obtained during manual operation, to a mean of 3.9 ppm for automatic operation. Figure 4 shows the effluent BOD data plotted on logarithmic probability paper to obtain a frequency distribution of measurements. The slope of the lines reflects the degree of reliability. For example, in Figure 4 the reduced slope of the automatic control line indicates that automation will result in tighter control of effluent BOD.

Further analysis indicated that the sludge characteristics may also have been affected by automatic DO control. The frequency distribution of the sludge volume index (SVI) is shown in Figure 5. The arithmetic mean for the SVI with manual control was 332. This was reduced to 86 with automatic control. The difference in the slopes of the two lines is more marked in this case (Figure 5) indicating a greater advantage for automatic control in maintaining a consistent product.

At Palo Alto the computer calculated the DO setpoints using data obtained from DO probes and then alerted the operator to make the required change. In contrast during the manual operation, the operator monitored the DO twice per shift and then made the appropriate corrections. When the semi-automatic operation was compared to manual operation (Figure 6), a 13% performance improvement as measured by effluent TOC and an 11% reduction in air use was observed. The latter calculates to a $5,380 savings per year for a 25 mgd plant. When a cumulative frequency plot is made of manual vs. DO control, the slopes of the lines were equal indicating that the reliability of both systems was similar. This is understandable since the final control implementation was carried out by the same operators.

The other control strategies that were evaluated at Palo Alto concentrated on regulation of food-to-microorganism ratio (F/M). Several techniques such as TOC, COD and oxygen uptake were considered for measuring the food. However, suitable automatic TOC and COD analyzers were not available for on-line control during the Palo Alto experiments. Therefore, only two F/M control strategies could be evaluated. These were a) feedback respirometry control using an on-line respirometer, and b) DO/RAS control. In both cases the DO was controlled as described previously and since the results were similar only DO/RAS control loop will be described.

For DO/RAS control, the rate of air demand was assumed to be proportional to the BOD input and the return sludge was adjusted to maintain the desired F/M ratio. In other words, the entire aeration tank was used as a respirometer to set the return sludge flow.

In all these evaluations, the system was allowed about 30 days to stabilize, and then, three days of intensive sampling was carried out. Figure 7 shows the BOD loading (food uptake) that was measured every two hours. Using the aeration tank as a respirometer the food uptake was also estimated and compared to the BOD loading. Under manual operation, that is, maintaining a constant RAS, the mixed liquor suspended solids (MLSS) varied widely as shown in Figure 8. When DO/RAS control is employed, the MLSS becomes more constant (Figure 9). However, when comparing these results to those obtained when only DO control was used the plant showed no performance improvement in terms of effluent quality or cost savings.

The results do indicate that DO control is a valuable control loop that should be explored further. The ease and simplicity of installing and maintaining a DO loop is more than compensated for by the cost savings and performance improvements. The case for F/M control however, is not so conclusive. Because of fund limitations, the evaluations at Palo Alto were limited to about 30 days each. There is some speculation that had more time been allowed, especially for manual evaluations, more significant results could have been obtained.

AUTOMATION OF PHYSICAL-CHEMICAL TREATMENT SYSTEM

The basic physical-chemical process sequence that was evaluated at Blue Plains (4) is shown in Figure 10; it consists of two-stage (high pH) lime precipitation with intermediate recarbonation, dual-media filtration, pH control with chlorine, CO_2 stripping, breakpoint chlorination and

FIGURE 4 FREQUENCY DISTRIBUTION OF BOD IN THE EFFLUENT

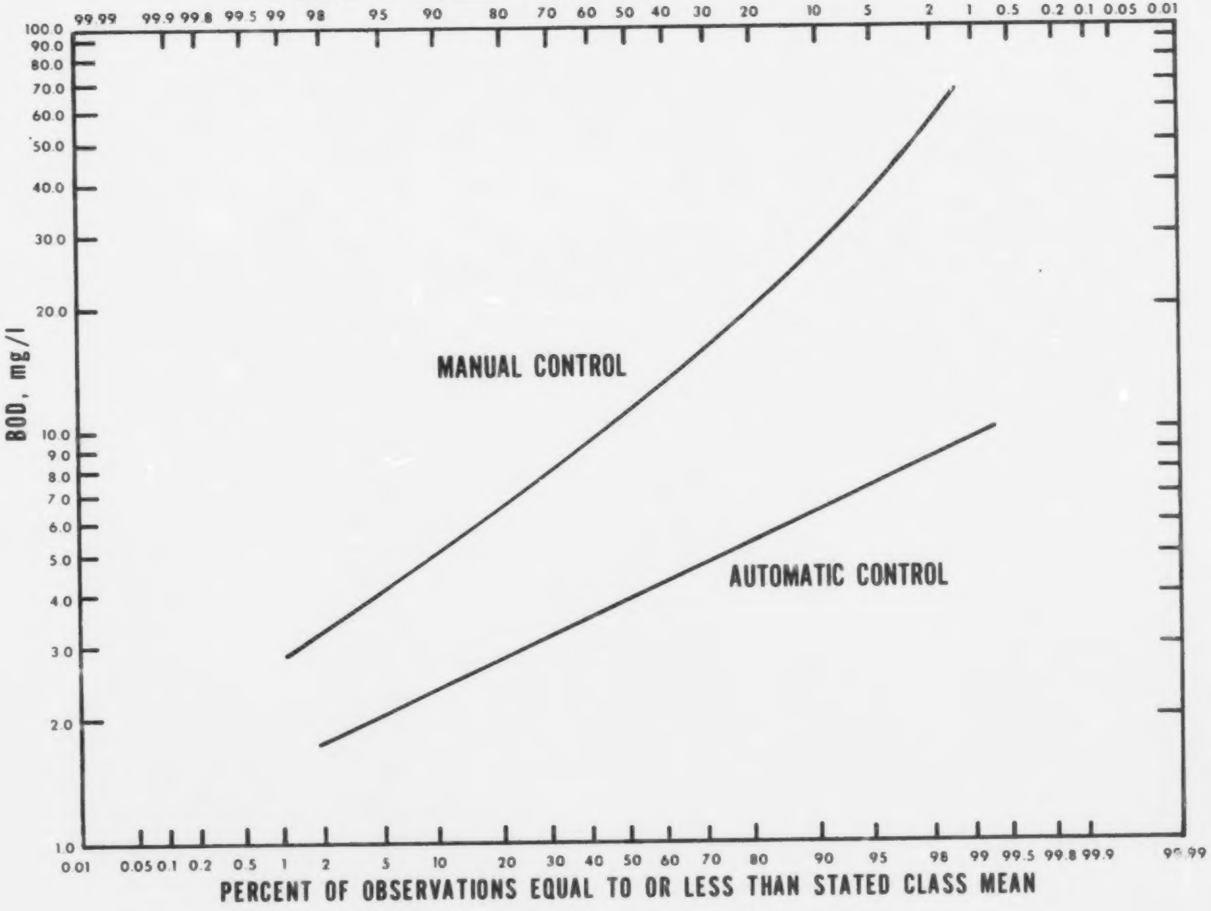
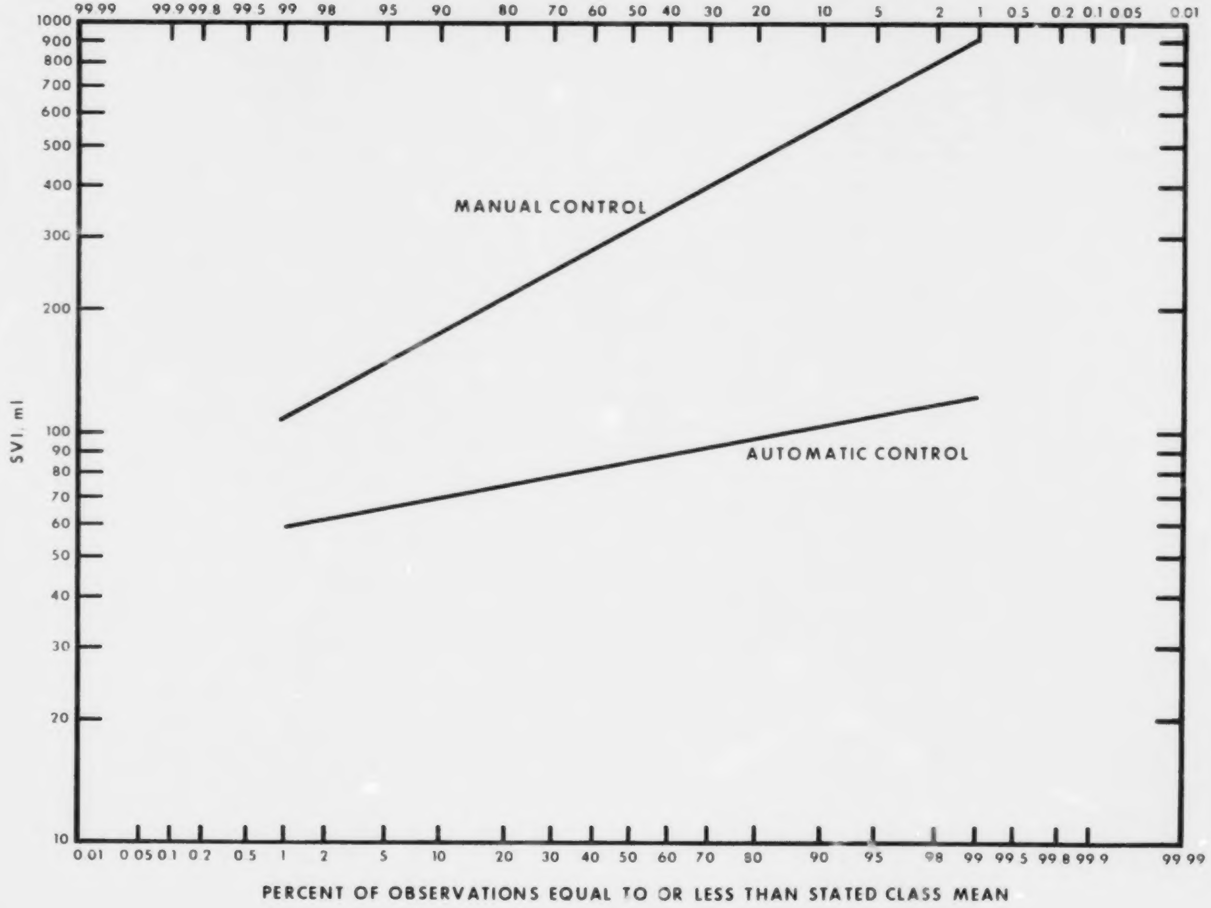


FIGURE 5 FREQUENCY DISTRIBUTION OF SVI



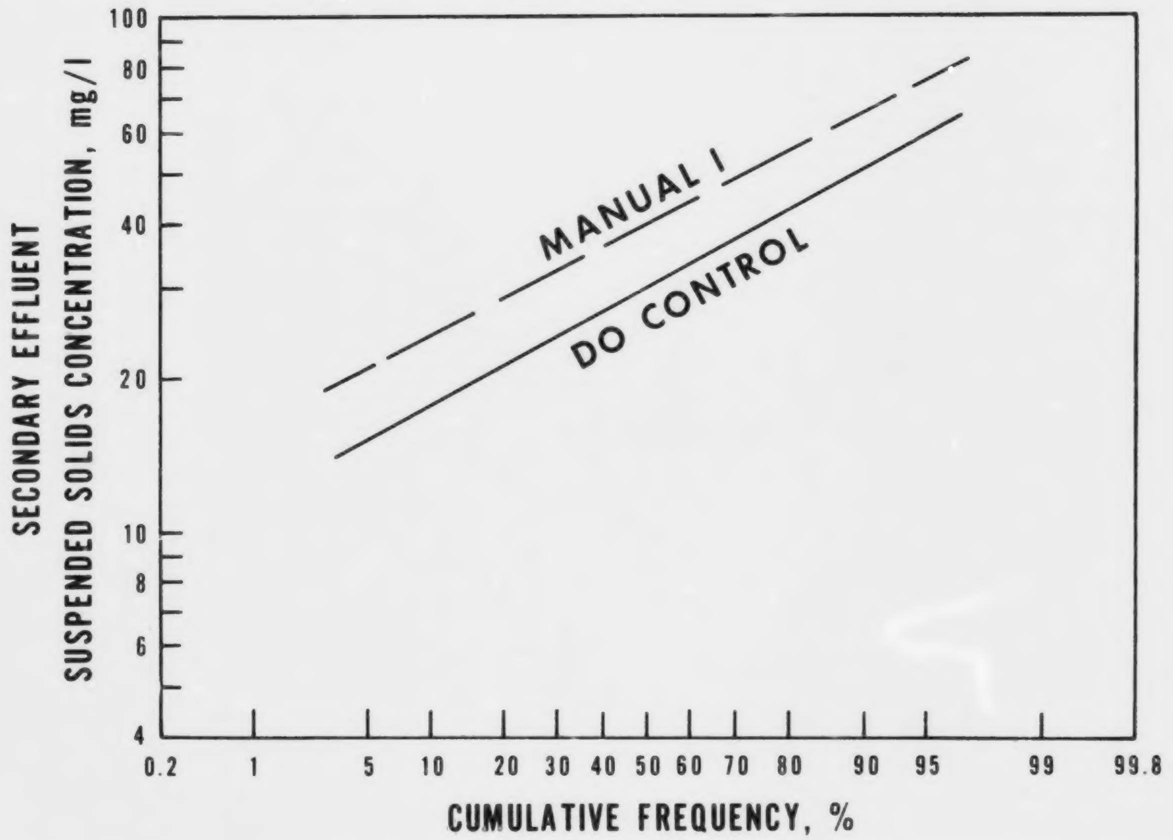


FIGURE 6. DRY SEASON TESTS: SECONDARY EFFLUENT SUSPENDED SOLIDS CONCENTRATION VS. LOG NORMAL FREQUENCY DISTRIBUTION

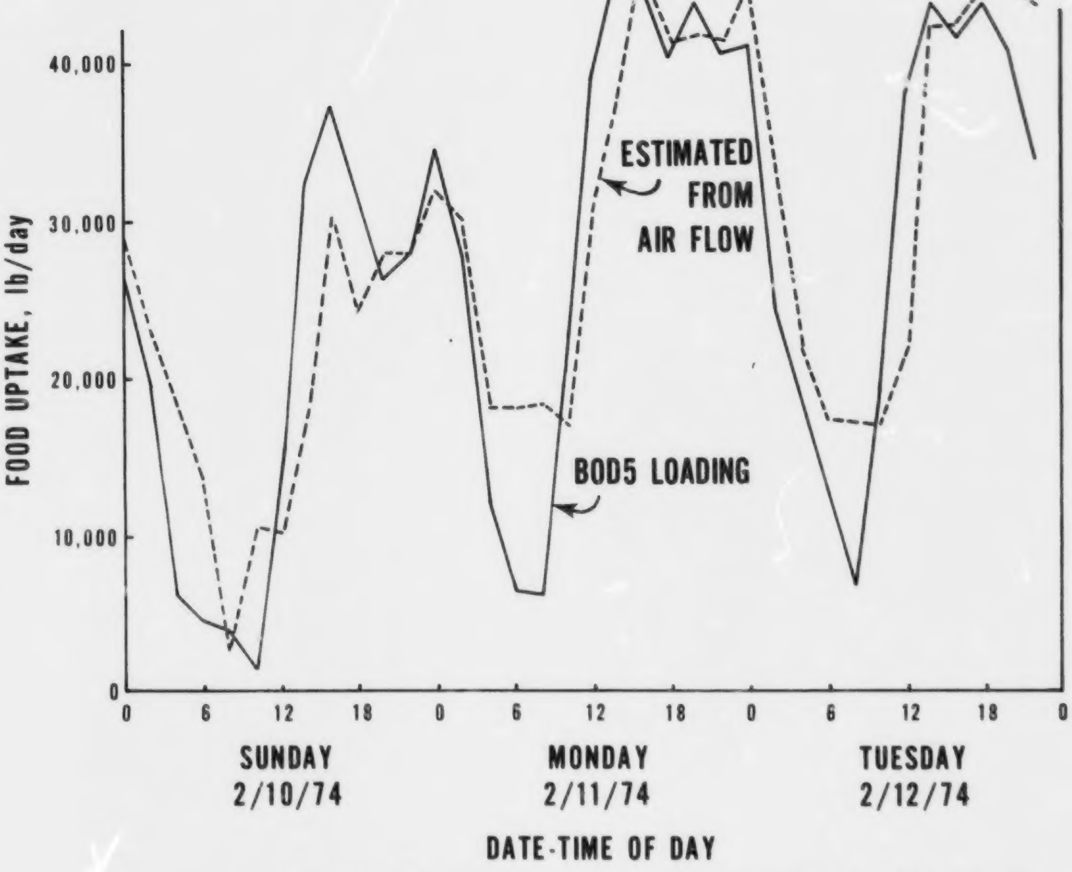


FIGURE 7 . AIR/RAS CONTROLLER: ACTUAL AND ESTIMATED FOOD UPTAKE

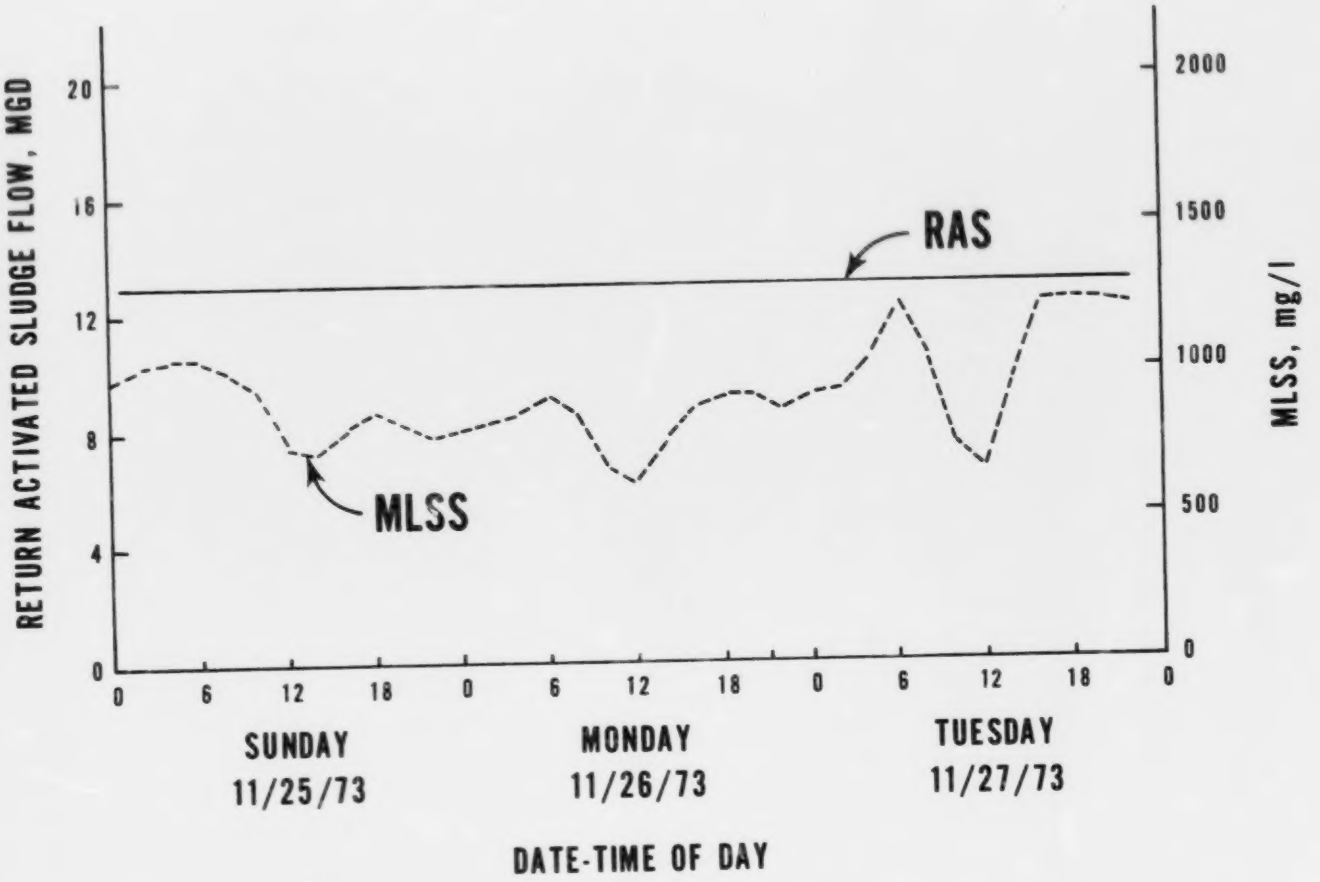


FIGURE 8 MANUAL TEST II: MLSS AND RAS VS. TIME

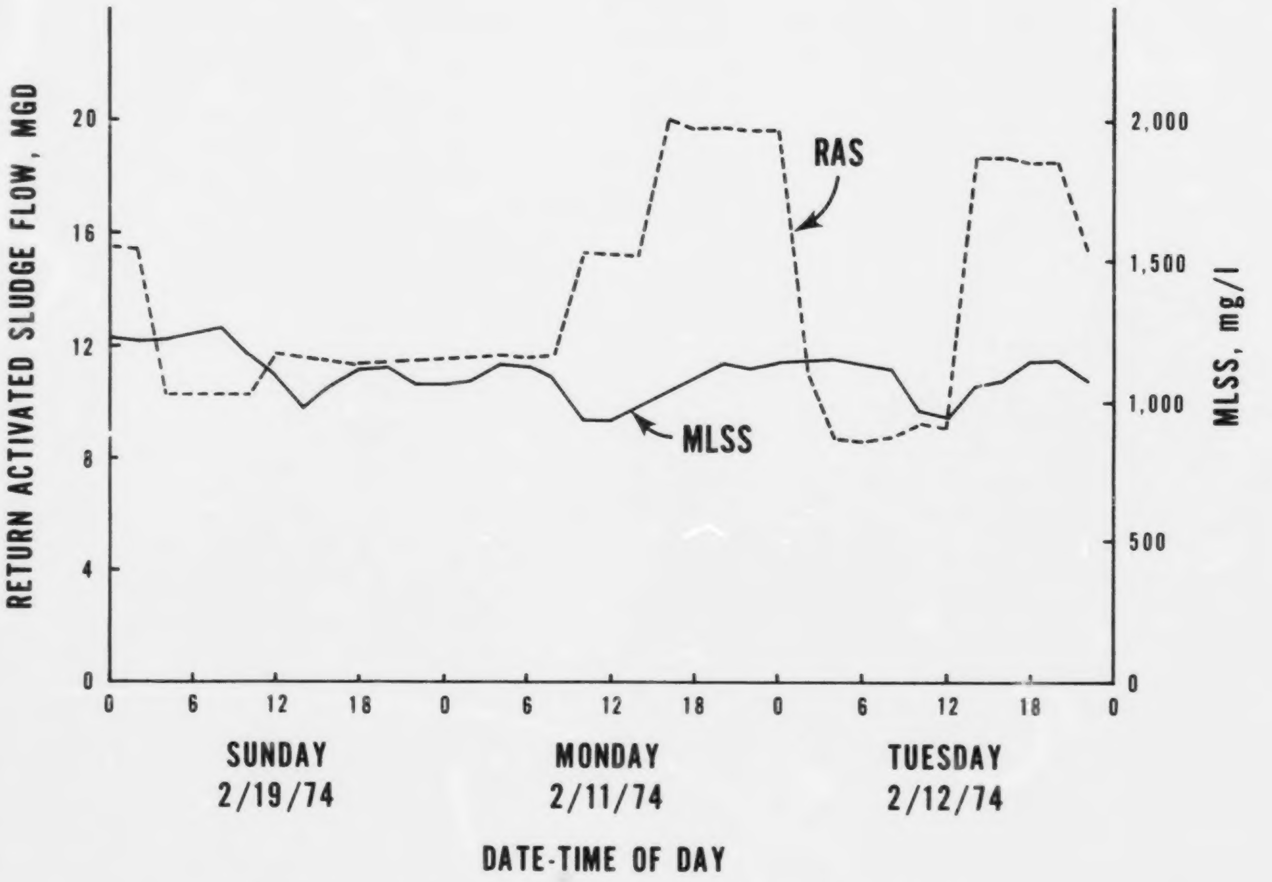


FIGURE 9. AIR/RAS CONTROLLER: MLSS AND RAS VS. TIME

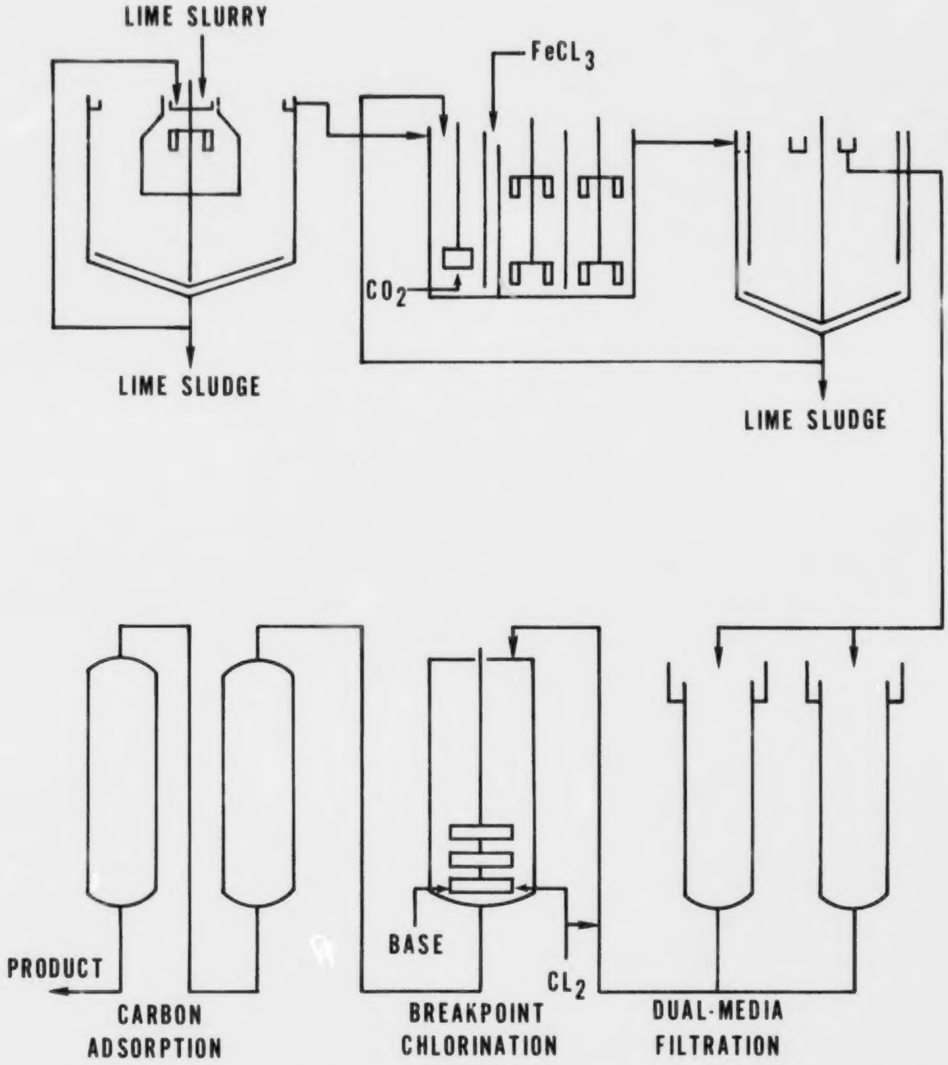

LIME SLURRY
FeCL$_3$
CO_2
LIME SLUDGE
LIME SLUDGE
PRODUCT
BASE
CL_2
CARBON ADSORPTION
BREAKPOINT CHLORINATION
DUAL-MEDIA FILTRATION

FIGURE 10

PHYSICAL CHEMICAL TREATMENT

TWO-STAGE CHEMICAL CLARIFICATION
WITH INTERMEDIATE RECARBONATION

granular carbon adsorption. In the first stage of the process, powdered lime, raw wastewater, and recycled solids are rapid-mixed, flocculated and settled to remove bicarbonate, phosphate and magnesium. The pH of the clarified effluent is reduced from 11.5 to 9.5 in a recarbonation tank. The effluent from the second stage clarifier flows by gravity through dual-media filters consisting of coal and sand. After filtration, chlorine is added in a static mixer to reduce the pH to 6-7. Carbon dioxide is air stripped from the filter effluent to reduce the alkali required for the breakpoint reaction. Breakpoint is achieved by adding chlorine and alkali to the wastewater ahead of a second static mixer and contact tank. The base is added to control the pH and prevent formation of undesirable end products such as nitrate and nitrogen trichloride. Soluble residual organics are removed from the wastewater with two-stage downflow carbon columns using a contact time of 40 minutes. A high quality effluent is produced by this treatment sequence. Typical water quality includes 5 mg/l of BOD, 15 mg/l of COD, 0.15 mg/l of P and 2.6 mg/l of N (3).

Lime Clarification

A dry lime feeder maintains a constant-concentration lime slurry which is circulated through a head tank above the primary reaction zone of the clarifier. An electro-pneumatic valve meters the slurry into the reaction zone on demand signals from the lime-feed controller. The four alternative strategies studied for lime-feed control were 1) conductivity-ratio, 2) flow-proportional, 3) pH plus flow-proportional, and 4) alkalinity plus flow-proportional.

Figure 11 schematically represents the basic components used in the four alternative control schemes. The conductivity-ratio control scheme involves the measurement of conductivity in the primary reaction zone and in the influent wastewater. The ratio of these conductivity measurements generates a control signal (C) for the lime-feed valve. For flow-proportional control, the influent flow rate is measured, and this signal (properly modulated at MT and PS) is transmitted directly to the control valve. For pH plus flow-proportional control, the pH is measured in the primary reaction zone, and this signal is used to adjust the signal generated from the flow-proportional loop. For alkalinity plus flow-proportional control, a sample is pumped from the clarified zone of the clarifier through a porous rock filter to an automatic titrator. The resulting alkalinity signal is transmitted to the multiplying transmitter in a flow-proportioning control system for final adjustment of lime addition.

The results of a seven-day test run are shown in Table 1.

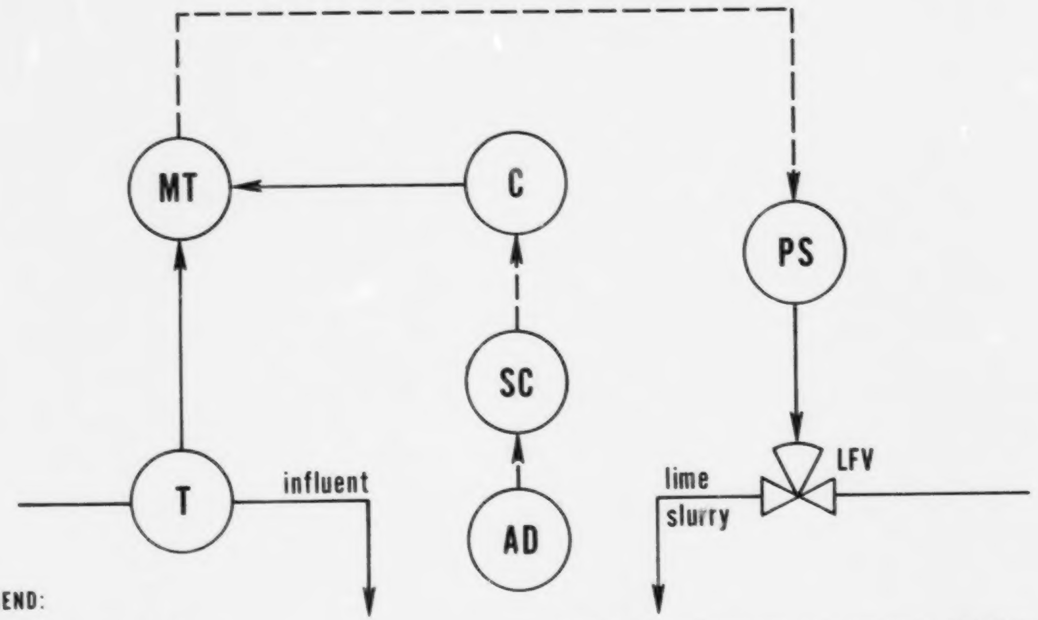
TABLE 1. PERCENTAGE DEVIATION FROM TARGET DURING SEVEN-DAY TEST RUN

Control Scheme	Ranges of Deviation from Target Alkalinity, %
Conductivity-ratio	+ 16 to -20
Flow-proportional	+ 15 to -15
pH plus flow-proportional	+ 10 to -10*
Proportional	+ 10 to -15†
Alkalinity plus flow-proportional	+7.5 to -7.5

*1st 2 days
†Entire 7-day test period.

Conductivity-ratio control was found to be the least accurate control system, but it would be a good backup control system since it is dependable and the equipment requires little maintenance. The flow-proportional control system was very sensitive to any change in lime-slurry concentration and was very much dependent on the accuracy of the flow measurement device. After seven days of operation, the pH electrodes were coated with a calcium carbonate scale which was approximately 1/16-inch thick. This coating was removed in 2% hydrochloric acid, and the electrode regained its initial response characteristics. By scheduling electrode cleaning every 2 days, pH control will work satisfactorily. Placement of the pH probe in a separate rapid-mix tank reduces the maintenance requirements associated with placement in the primary reaction zone of a single-unit clarification system.

While alkalinity plus flow-proportional control produced the closest alkalinity control of all the systems studied, the



LEGEND:

T = Reference Transducer
MT = Multiplying Transducer
C = Controller
SC = Signal Conditioner
AD = Measuring Transducer
(Analytical Device)
PS = Pulse Transmitter

REQUIRED COMPONENTS FOR CONTROL SCHEMES

TYPE OF CONTROL	COMPONENT
CONDUCTIVITY RATIO	T , AD , C
FLOW-PROPORTIONAL	T , MT , PS
pH PLUS FLOW-PROPORTIONAL	T , AD , SC , C , MT , PS
ALKALINITY PLUS FLOW-PROPORTIONAL	T , MT , AD , PS

FIGURE 11
LIME FEED CONTROL SCHEMES

equipment malfunctioned repeatedly because of filter clogging. The inability to filter high solids concentrations efficiently required relocation of the sample point from the reaction zone to the clarified zone. This resulted in a two-hour lag in the response time which caused large swings in process effluent quality when the lime-slurry concentration changed. Until the solids handling problem for alkalinity plus flow-proportional control is solved, the recommended control system is pH plus flow-proportional, with conductivity-ratio control as a backup.

The solids wasting control loops for the first and second stage clarifiers are simple feedforward systems where periodic pulses, proportional to flow, produce a discharge. The discharge from this tank is controlled by a level switch sensing the fixed volume.

The control loop for the $FeCl_3$ feed in the recarbonation tank is a feedforward flow-proportional system which changes the duty cycle of the dosing pump.

The carbon dioxide dosing control system is the same pH plus flow-proportional control system used for the lime addition control. The chemical dose is controlled by a feedforward signal proportional to flow and a feedback signal generated by pH error.

Filtration

Operation of the dual-media gravity filters was controlled with four, alternative, backwash initiation and control schemes. Alarm schemes used to initiate backwash had time-delay circuits to prevent accidental or momentary events from triggering the backwash cycle prematurely. The four models used were 1) headloss, 2) high-level (influent level), 3) programmed time-interval, and 4) manual. The headloss sensor initiates the backwash cycle when the available head decreases to a preset minimum valve (H.L. = 9 ft H_2O). When the level tends to change, the high-level indicator opens an effluent control valve so that a constant level is maintained. When the control valve is 100% open, backwash is initiated. The programmed time-interval controller will initiate backwash at the expiration of a preselected number of operating hours. The operator may over-ride any of the above controls at any time with the manual mode.

The effluent from clarification was distributed equally to the operating filters by a mechanical splitter box. As a filter was isolated for backwash, the flow to that filter was redistributed to the remaining operating filters. If the headloss alarm was used and if the filter backwash occurred at peak flow rates, the redistribution caused the already stressed operating filters to be overstressed. The final result was a chain reaction resulting in the need to backwash all available filters in a relatively short period of time which increased the requirements for backwash-water pumps and storage capacity.

The programmed time-interval controller was used to schedule filter backwashing at different hours during periods of low flow; this reduced backwash-water pumping and storage requirements, and it eliminated overstressing of the system. The headloss indicator was then used as a backup alarm to prevent flooding when system upsets caused increased solids loading and shorter filter runs than the programmed time interval. The high-level alarm was connected to an audio-visual alarm and was used to indicate equipment failure. This system has provided peak operating efficiency at the lowest possible operating cost.

Breakpoint Chlorination

The control scheme developed to control the breakpoint-chlorination process is shown schematically in Figure 12. The chlorine dosage-control loop employs a feedforward signal proportional to the mass of influent ammonia and a feedback signal based on the free residual chlorine concentration error. The feedforward signal is derived from the concentration of ammonia in the influent, the influent flow rate, and a preselected weight ratio of chlorine to ammonia. If digital control of the system were practiced, this feedforward signal would be adjusted by the amount of chlorine used for pH control during prechlorination. The control loop for alkali addition (NaOH) is derived from a feedforward signal based on the chlorine dosage used and a feedback signal based on the pH error. The on-stream analysis of ammonia by a chlorimetric analyzer,

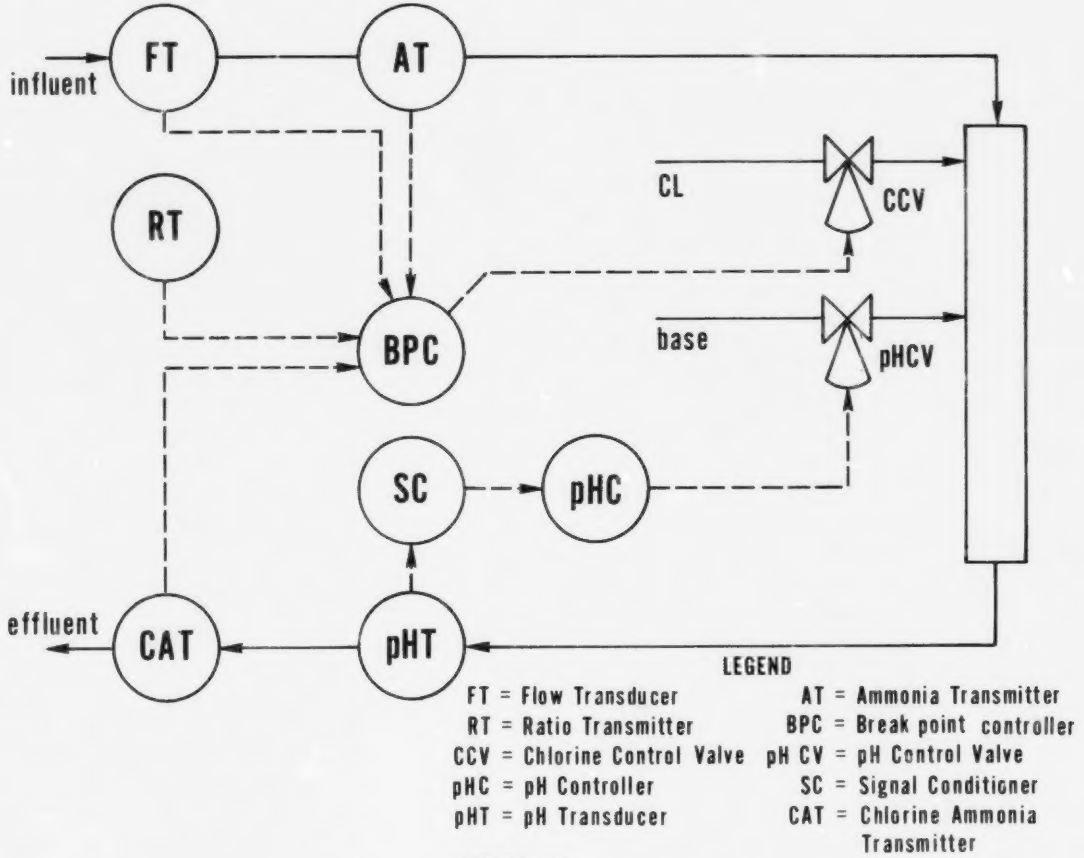

influent
FT
AT
RT
BPC
CL
CCV
base
pHCV
SC
pHC
effluent
CAT
pHT
LEGEND
FT = Flow Transducer
RT = Ratio Transmitter
CCV = Chlorine Control Valve
pHC = pH Controller
pHT = pH Transducer
AT = Ammonia Transmitter
BPC = Break point controller
pH CV = pH Control Valve
SC = Signal Conditioner
CAT = Chlorine Ammonia Transmitter

FIGURE 12

BREAK POINT CHLORINATION CONTROL

both before and after breakpoint chlorination, has been accurate and dependable. Free residual chlorine is also continuously measured by a colorimetric analyzer. Preliminary operating experience has been favorable. Breakpoint chlorination reduces operating problems with the carbon adsorption system by reducing biological slime growths. The carbon adsorption process is a good backup system for the breakpoint chlorination process because of the dechlorination potential.

Carbon Adsorption

The carbon adsorption process presented control problems very similar to the filtration system; however, the carbon adsorption process was only semi-automated. A level-controller regulated the flow through the system, while an automatic pressure-controller on the discharge side of the carbon-column feed pump maintained a constant pressure at the inlet to the first carbon column. There were five carbon columns interconnected in series by headers and automatic valves to allow any number of columns to be operated in sequence and any column to be the lead column. Column sequencing and backwash initiation were determined by an operator. The lead column is backwashed daily in the raw wastewater-treatment applications.

Direct Digital Control

An IBM Systems/7 computer was installed at the EPA Blue Plains Pilot Plant to provide total systems control for the independent physical-chemical treatment sequence (5). The process-control computer also performed a data-acquisition function. Plant data from approximately 100 wastewater-process sensors were collected, converted into engineering units and stored for later analysis and evaluation of the systems performance.

Alarm modes were also included to alert the operator of plant conditions that require his attention and correction. Examples are out-of-range alarms for such parameters as pH and chemical doses to the wastewater, as well as equipment-failure alarms for such items as sludge-blowdown equipment and pneumatic pumps.

Direct digital computer control is being used more frequently in the larger wastewater treatment facilities where the economies realized in data acquisition, reporting, preventive maintenance scheduling, load programming and improved performance can offset the additional costs of specialized manpower and the capital costs of the initial hardware and backup system. The capability to control individual unit processes and indeed complete physical-chemical treatment systems is rapidly becoming a reality.

RESEARCH NEEDS AND PROBLEM AREAS

To initiate a coordinated attack on instrumentation and automation problems in this field, a workshop entitled Research Needs for Automation of Wastewater Treatment Systems (6) was held in Clemson, South Carolina in September 1974. This workshop, sponsored by the EPA in cooperation with Clemson University, provided an opportunity for workers in this area to discuss their research problems and needs.

The workshop found that the general problem areas were the lack of: adequate field experience, quantitative understanding of wastewater systems, and required sensors. Or to put it another way, the problems are a lack of sensors and of fundamental knowledge about the treatment processes. These problems were stated in almost every session. To resolve these problems, the needed research should include demonstrations of automated process control, development of mathematical models and algorithms, and evaluation of sensors. The workshop also indicated a need for an information clearinghouse, including the international exchange of data; and projected a new philosophy of wastewater renovation as opposed to processing wastewater to minimum quality requirements. The cost-effective application of instrumentation and automation to wastewater management systems will be a key to implementing this philosophy. One immediate outgrowth of the workshop was the establishment of an EPA Advisory Committee on Instrumentation and Automation for Wastewater Management. The functions of this advisory committee will be to:

- serve as focal point for questions concerning instrumentation and automation of wastewater management systems within the United States, and provide liaison with similar groups in other nations

- facilitate the exchange of research information in this area

- identify and characterize the technology required for instrumentation and automation of wastewater management systems

- assist the EPA in defining short- and long-range research development and demonstration programs

The roster of this advisory committee consists of four EPA members and eight non-EPA members. The latter represent state and local governments, the academic community, equipment manufacturers and professional societies. The communication resulting from such a widely represented group is desirable because it establishes a link between the user community, the researcher and enforcement agencies. This provides for the opportunity for each of these groups to rapidly communicate goals, problem areas and solutions with one another.

REFERENCES

1. Molvar, A.J., et al., "Instrumentation and Automation Experiences in Wastewater Treatment Facilities." EPA report being prepared for publication.

2. Roesler, J.F., "Plant Performance Using Dissolved Oxygen Control." Jour. Environ. Eng. Div., 100, 1069 (1974).

3. Petersack, J.F. and Smith, R.G., "Advanced Automatic Control Strageties for the Activated Sludge Treatment Process." EPA-670/2-75-039 (May 1975).

4. Convery, J.J., et al., "Automation and Control of Physical-Chemical Treatment for Municipal Wastewater." Applications of New Concepts of Physical-Chemical Wastewater Treatment, Sept. 18-22, 1972. Pergamon Press, Inc. USA (1972).

5. Bishop, D.F., et al., "Computer Control of a Chemical Clarification Waste Treatment Pilot Plant." Presented at the First International Meeting - Pollution: Engineering to Scientific Solutions, Tel Aviv, Israel (June 12-17, 1972).

6. Buhr, H.O., et al., Ed. "Research Needs for Automation of Wastewater Treatment Systems." Proceedings of a Workshop held at Clemson, S.C. September 23-24, 1974 (June 1975).

RESEARCH REQUIRED TO ESTABLISH CONFIDENCE IN THE POTABLE REUSE OF WASTEWATER

J.N. English, K.D. Linstedt, and E.R. Bennett
J.N. English, Sanitary Engineer, Wastewater
Research Division, Municipal Environmental Research
Laboratory, EPA, Cincinnati 45268; and K. D. Linstedt
and E. R. Bennett Associate Professors of Sanitary
Engineering at the University of Colorado
Boulder, Colorado 80302

ABSTRACT

A Municipal Research Needs Workshop, sponsored by the Environmental Protection Agency (EPA), the Water Pollution Control Federation (WPCF), and the American Water Works Association (AWWA), was held at the University of Colorado, Boulder, Colorado, in March 1975 to identify priorities for research that would provide scientific knowledge and technology to prove the feasibility and practicability of reusing wastewaters for potable purposes. Ninety-two select persons concerned with municipal wastewater reuse in the United States and abroad discussed and identified a long term research program involving; treatment technology; treatment reliability and quality control; health effects associated with organic, inorganic, and biological pollutants; and the socio-economic aspects of potable reuse.

BACKGROUND

Sound management of water resources must include consideration of the potential use of properly treated wastewaters as an alternate means of meeting future water demands. Consumptive water use data indicate that the lower Colorado River region uses more water than its available natural supply, and the Great Basin and Rio Grande regions use 60 percent or more of their average supplies (1). Large, economically important regions of the Nation already are, or will be using water beyond the capacity of the available natural water resources. Steadily increasing municipal and industrial water requirements in these areas, combined with expanding irrigation activities, could place severe strains upon limited water resources. At the same time, many areas are facing the growing problems of water quality deterioration.

Groundwater in many areas is being mined or used at rates exceeding recharge capability. The present economy of these areas is based upon the foundation of a temporary and dwindling water resource. In major groundwater-using areas, such as Lond Island and Southern California, substitute supplies can be obtained only at relatively high cost. Where water development costs and legal constraints are making sources of raw water difficult to acquire, wastewater reuse is an attractive alternative water source. In addition, reuse is an effective method of dealing with water pollution by preventing degradation of water quality.

Public Law 92-500, the Federal Water Pollution Control Act Amendments of 1972, recognizes the potentially large benefit to be realized if wastewaters can be renovated for reuse applications. The Safe Drinking Water Act of 1974 also contains mandates of importance with regard to renovation and recycling of wastewaters. In particular, Section 1444 authorizes a development and demonstration program to: (1) investigate and demonstrate health implications involved in the reclamation, recycling, and reuse of wastewaters for drinking; and (2) demonstrate processes and methods for the preparation of safe and acceptable drinking water from wastewaters. There exists, therefore, a strong and clear legislative mandate for research, development, and demonstration of reliable, cost-effective technology for reclaiming and recycling wastewaters for beneficial uses. A major beneficial use is the supplementation of domestic water supplies.

Advanced wastewater treatment systems developed primarily for pollution control are available for removing pollutants to very low levels. As a result of this technology, and the need for additional sources of water to meet present and future requirements, renovated wastewater is being considered in planning for overall water resources utilization in many areas of the country. The treatment technology capability can be identified for reuse applications such as agricultural, recreational, and industrial where quality requirements have been defined. The State of California has already established water quality standards for wastewater used for agricultural and recreational purposes (2). Since there are public health questions in this country concerning the degree and reliability of treatment for potable reuse, the necessary treatment technology is less well defined for this use than for other types of reuses.

Because of the limited experience with direct or overt potable reuse of renovated wastewater there are no standards to apply to such waters. The U.S. Public Health Service (USPHS) Drinking Water Standards of 1962, and the Environmental Protection Agency (EPA) Proposed Interim Primary Drinking Water Standards apply to water sources that are as free as possible from pollution. The concern among many in the water field over the uncontrollable source and characteristics of wastewater has resulted in the recommendation that renovated wastewater should meet stringent standards in addition to those written for unpolluted sources. This concern would appear to be reasonable, but should not be restricted to wastewater since many surface water supplies do not qualify as "unpolluted."

Health effects research relating to wastewater reuse has not paralleled the developments in wastewater treatment technology. The quality of present conventional water supplies has only been questioned rather recently relative to the trace contaminants. Health effects studies are presently underway on approved sources of potable water. However, much remains to be done since many problems, such as the presence of organic materials and their potential health hazards in drinking waters, have not been solved. This needed research will be applicable to wastewater reuse since the production of water from surface supplies that contain a significant portion of effluents is analogous to the direct recovery of water from wastewater.

In addition, research to determine the economic, social, and political aspects of potable reuse has not kept pace with treatment technology development. The EPA has only recently identified needed information in this area, and this has come about as a result of the potential availability of highly treated wastewaters that are "too good to throw away." These waters have attracted the attention of those agencies in water-short areas that are responsible for maintaining adequate supplies of drinking water.

RESEARCH NEEDS WORKSHOP

The Water Pollution Control Federation (WPCF) and the American Water Works Association (AWWA) issued a joint resolution that urged the Federal Government to support a massive research effort to develop needed technology for the potable reuse of wastewater. These organizations underscored the "lack of adequate scientific information about possible acute and long-term effects on man's health from such reuse," and noted that "essential fail-safe technology to permit such direct reuse has not yet been demonstrated." The resolution recognized the need for an "immediate and sustained multi-disciplinary, national effort to provide the scientific knowledge and technology relative to the reuse of water for drinking purposes in order to assure the full protection of the public health."

The EPA Office of Research and Development (OR&D), through its Municipal Pollution Control Division, has devoted effort for a number of years to the development of wastewater treatment processes capable of producing high quality effluents suitable for a range of reuse or recycling applications. More recently the OR&D Health Effects and Socio-Economic Research Programs have initiated research related to potential health problems and the social-economic impact of reusing wastewater.

Because of legislative mandates and the need to include practical applications of wastewater reuse in the management of water resources, EPA has intensified its efforts to identify research needs as a first step in establishing a more viable wastewater reuse program. To implement a multi-disciplined research effort a method was needed for identifying research priorities and providing EPA with direction for conducting the municipal wastewater potable reuse research program. The workshop method was selected as the best approach to defining and establishing priorities for potable reuse research. Ninety-two persons concerned with wastewater reuse, from this country and abroad, were brought together to discuss and identify research gaps in the areas of health effects, treatment technology, and the socio-economic considerations of potable reuse.

The Workshop was jointly sponsored by the Environmental Protection Agency (EPA), the Water Pollution Control Federation (WPCF), and the American Water Works Association (AWWA), and was held at the University of Colorado at Boulder, Colorado on March 17-20, 1975.

The Workshop was three days in length and was designed for persons involved in the use, conduct, direction, or specification of research in the wastewater renovation and reuse field. WPCF and AWWA, and selected representatives of federal, state, municipal, industrial, academic, and consulting organizations participated. In addition to presentations on specific reuse situations that included both native and foreign experience, papers were presented on 1) potential health hazards of using wastewater for domestic purposes, 2) the status of existing technology that can be used to properly treat wastewater for reuse, and 3) the socio-economic aspects of reuse. The presentations established the state-of-the-art, or "where we are now."

The evening of the first day and the entire second day were devoted to separate workshop groups each dealing with one of the following specific topics related to potable reuse:

- ° Wastewater Treatment
- ° Treatment Reliability and Quality Control
- ° Health Effects Associated with Inorganic Pollutants
- ° Health Effects Associated with Organic Pollutants
- ° Health Effects Associated with Biological Pollutants
- ° Socio-Economic Aspects

Workshop participants were divided about equally between these groups according to their interest and experience. Each Workshop group had a prior designated chairman and vice chairman to guide the group discussions and summarize the problems identified and the research needed to solve these problems. The results of each group's efforts were presented to the total Workshop attendees on the third day for further discussion and multi-disciplinary comments.

IDENTIFIED RESEARCH

The research needs associated with the potable reuse of municipal wastewater as identified by the six individual Workshop groups are presented separately as follows:

1. Treatment - Characterization of the effectiveness of alternate wastewater treatment systems for the removal of organics, trace metals, nitrogen, bacteria, viruses, parasites, and suspended material was considered to be the highest priority research need. The four treatment trains shown in Figure 1 were identified as having potential for reliably producing potable quality water from raw (primarily domestic) wastewater. A need was identified to evaluate these systems for energy requirements, costs, by-product production, overall reliability, and removal of the pollutants previously mentioned. It was estimated that the intensive characterization period would require at least three years, plus any further time needed for evaluation of

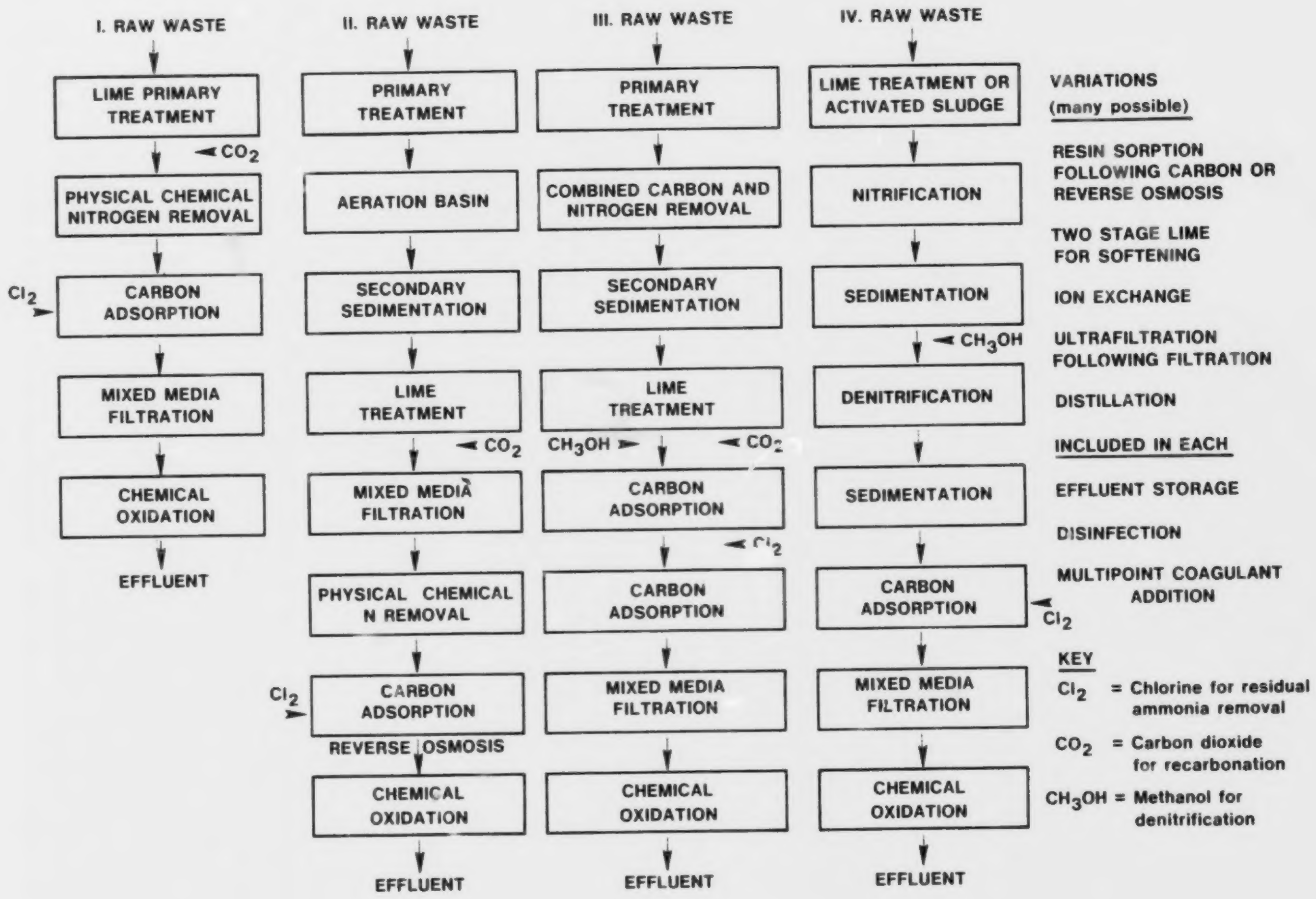

I. RAW WASTE
LIME PRIMARY TREATMENT
CO₂
PHYSICAL CHEMICAL NITROGEN REMOVAL
Cl₂
CARBON ADSORPTION
MIXED MEDIA FILTRATION
CHEMICAL OXIDATION
EFFLUENT
II. RAW WASTE
PRIMARY TREATMENT
AERATION BASIN
SECONDARY SEDIMENTATION
LIME TREATMENT
CO₂
MIXED MEDIA FILTRATION
PHYSICAL CHEMICAL N REMOVAL
Cl₂
CARBON ADSORPTION
REVERSE OSMOSIS
CHEMICAL OXIDATION
EFFLUENT
III. RAW WASTE
PRIMARY TREATMENT
COMBINED CARBON AND NITROGEN REMOVAL
SECONDARY SEDIMENTATION
LIME TREATMENT
CH₃OH
CO₂
CARBON ADSORPTION
Cl₂
CARBON ADSORPTION
MIXED MEDIA FILTRATION
CHEMICAL OXIDATION
EFFLUENT
IV. RAW WASTE
LIME TREATMENT OR ACTIVATED SLUDGE
NITRIFICATION
SEDIMENTATION
CH₃OH
DENITRIFICATION
SEDIMENTATION
CARBON ADSORPTION
Cl₂
MIXED MEDIA FILTRATION
CHEMICAL OXIDATION
EFFLUENT
VARIATIONS
(many possible)
RESIN SORPTION FOLLOWING CARBON OR REVERSE OSMOSIS
TWO STAGE LIME FOR SOFTENING
ION EXCHANGE
ULTRAFILTRATION FOLLOWING FILTRATION
DISTILLATION
INCLUDED IN EACH
EFFLUENT STORAGE
DISINFECTION
MULTIPOINT COAGULANT ADDITION
KEY
Cl₂ = Chlorine for residual ammonia removal
CO₂ = Carbon dioxide for recarbonation
CH₃OH = Methanol for denitrification

FIGURE 1. VARIOUS POTABLE WATER TREATMENT PROCESS TRAINS

health effects. It was established that each system should be demonstrated on at least a 0.5 mgd (1890 cu.m./day) scale to establish performance credibility, and that the systems should be applied to municipal wastewaters, domestic wastewaters, and polluted river water.

System I is an independent physical-chemical process. System II is a secondary-tertiary process. System III is similar to System II except that the bulk of the nitrogen is removed in the initial one-step biological treatment stage. System IV is an integrated chemical-biological treatment system. Rough costs were estimated to be (1975 dollars): $3,000,000 construction cost per system. Analytical and operation costs (excluding health effects) $800,000/yr. per system.

Disinfection is a mandatory requirement for potable wastewater reuse to insure protection of the public from infection. This requires the disinfection process to have a high degree of control and reliability with essentially no allowance for short term process failure. Four subcategories required to better direct the research in disinfection include: 1) examination of the by-products of all disinfection processes, 2) delineation of processes that can produce safe residual levels of disinfectants, and 3) identification of the best points for application of disinfectants in the treatment system.

Present technology for organic removal results in product waters containing 1 to 2 mg/l of total organic carbon. Although it has not been determined that there is a health hazard at this concentration, a need was identified to find processes to reduce the materials to lower levels. Potential methods that should be studied are chemical oxidation; resins; membranes; and volatile stripping. New methods and ideas for removing trace organics should be solicited from appropriate sources.

The identification and validation of indicators of system performance was identified as being necessary to permit monitoring without developing a burdensome analytical load. This will be required for reliable and cost-effective operation. An example of the indicators would be the use of turbidity and chlorine residual measurements as indicators of virus inactivation.

It was concluded that nitrogen removal will be required, but that currently available physical-chemical and biological nitrogen removal processes will need review because each has disadvantages. Modifications of these processes should be investigated to reduce their overall costs.

Although processes are available for removing many of the trace metals of concern from wastewater, selenates, arsenates, chromates, and molybdates are not well removed by most of the common AWT processes. Further study of potential removal processes is required. Demineralization for reducing salinity is a technique that has a beneficial effect on the removal of these ions, but the various demineralization processes need evaluation and improvement to reduce costs and energy requirements.

Two essential factors in reuse - quantity and quality - are impacted by storage. Storage may occur both inplant (recycle streams), and before or after the renovation facility. The main concern is finding the least costly method of providing the storage since the benefits of flow equalization and product protection are clearly evident.

Demonstration of methods for stabilization, recovery, and disposal of residuals is needed to insure the availability of adequate and economically viable control options for the accumulated residuals emanating from reuse treatment processes. This problem area is not unique to potable reuse, but it was identified as an important factor for eventual implementation of specific reuse treatment methods.

2. Treatment Reliability and Quality Control - A need was identified to define drinking water standards that can be applied with confidence to potable waters derived from polluted sources. The type of treatment systems and their reliability requirements will depend on the standards set for the produce water. Fail-safe reliability must be defined and the factors and procedures must be established which will assure that the product water

from a reuse system will be withheld from use if all quality criteria are not met.

The overall performance of a reuse treatment system may be affected by: 1) the time-dependent changes in the mass flow of some influent constitutents; 2) the presence or absence of unknown industrial constitutents; 3) the recycling of side streams; and 4) inplant chemical additions which may result in chemical reaction by-products. Therefore, information is needed on effluent variability and process reliability as they are affected by influent variability, the reliability of systems treating only domestic wastes vs. those treating combined domestic and industrial wastes, and the effects of recycle streams.

Some contaminants in domestic wastewater have been or will be identified as having an acute or chronic health effect when present in varying concentrations in reused wastewater. The cost of removing these contaminants in reuse systems will vary depending on the variability and reliability of removal required. Effective design and operation of reuse systems dictates that a level of "risk" be established that identifies the permissible variation in the removal efficiency of contaminants, and balances this with the cost of complete contaminant removal.

Once specific reuse systems are chosen, they should be evaluated to determine if there is a need for new types of equipment not now available. The needed reliability of such equipment should be determined and design specifications developed. As new products or equipment become available, their impact on potable reuse systems should be assessed and the information disseminated to the water industry. The rapid expansion of technology will bring new pollutants into the domestic and industrial wastewater streams that require continual updating of removal technology.

The key to product quality control in a reuse system lies in the availability of suitable monitoring systems. It is imperative that a few specific indicator parameters be identified and that automated monitoring equipment be developed so that the parameters can be correlated with the contaminants that may be health hazards.

Potable reuse systems are likely to be very complex. Step by step operation and control strategies with contingency action plans are needed in the event of process failures. Research is required to determine the frequency of failure of key pieces of equipment in various unit processes. Preventive maintenance procedures for both process equipment and instrumentation are needed to extend the life and increase the reliability of major reuse equipment. Increased operator capability and effective management are necessary to efficiently and reliably operate the complex treatment systems. Studies are needed to define the minimum plant size that is amenable to reliable operation, and to determine the staffing and organizational requirements to insure consistent operation.

Before successful reliability design can be achieved it is necessary to determine the allowable contaminant variability as well as monitoring and sampling frequency to insure adequate process control for providing the reliable removal of specific contaminants.

3. Health Effects/Inorganics - Research on this topic is intimately related to other environmental health research, especially to that relevant to drinking water criteria. This research is also related to ongoing studies on categorical diseases such as cancer and heart disease, and to geographical gradients in morbidity and mortality rates.

There is a need to compile data on the occurrence of inorganic materials in water and wastewater in those geographic areas where potable reuse is most likely to occur first. Multi-elemental analytical techniques such as spark source mass spectrometry, x-ray, fluorescence emission, and neutron activation should be used. Research is needed to perfect sample preparation techniques in order to increase the sensitivity, accuracy, and precision in monitoring.

Organometallic compounds and metal chelates are widely used in industry and have a high probability of occurrence and build-up in wastewaters. Little is known of their health effects, and identification is difficult. Examples include alkyl and aromatic amines, and dicarboxylic acid compounds that form complexes with metal cations.

The mere presence of an element or compound in water need not imply that there is any appreciable uptake. The material may be in an insoluble form, complexed, or chelated. Both animal and human studies, along with improved physical-chemical characterization are needed to determine the uptake fractions for various forms of inorganic constituents in water. Little is known of the role of particulate matter and its influence on uptake.

Populations consuming water of varying qualities should be examined from an epidemiological standpoint to determine whether specific contaminants represent a potential health effect. Body burden estimates can be based on medical research from samples of scalp hair, nails, blood, urine, subcutaneous fat, and deciduous teeth. Body burden changes can be detected more readily and much earlier than manifest toxicity. Hence, a greater level of health protection is implied than if one awaited more serious effects. Where cities are known to have differences in water constituents, autopsy, placental tissue, or surgical specimens may show that different water constituents lead to different concentrations of these same constituents in target organs. It may be possible to relate high levels of storage to morbidity and mortality from various causes.

In-vitro bacterial and cell culture systems should be used for the study of contaminants in water as: 1) primary toxicity screens; 2) indicator systems for determining active fractions in various fractionation schemes and refractionation of the active portions, and 3) systems for study of the interactions of chemicals for toxicity including mutagenesis and carcinogensis.

Conventional animal toxicity studies use relatively healthy normal animals, yet human health effects of greatest concern involve effects on infants, women during pregnancy, and on those with cardiovascular diseases. There are no well established relationships between results on healthy animals and those with impairment. Accordingly, studies are needed of selected types of animal systems to obtain toxicity data relevant to high risk human groups.

Behavioral toxicology has been shown to be a useful test system for effects of nitrites in rats. It has been shown to be a very sensitive indicator of the effects of certain compounds and is probably the only way alterations in cerebral function by toxic agents can be approached in animal studies.

The amount of inorganic materials in water does not necessarily relate to the amount ingested, since a high proportion of the liquid intake is from food and beverages. Therefore, it is necessary for each substance and class of substances to estimate the population dose based on usual food, beverage, and water ingestion.

Present epidemiological data relating death rates to components of drinking water, including components resulting from indirect or covert reuse of wastewater, are confused and uncertain. There is a need to obtain information on the chemical content of drinking water in metropolitan areas that exhibit a wide range of death rates. Studies should be made on the degree of association and correlation between these data and the age-sex-race-specific death and disease rates, particularly for age groups between 35 and 74 years, for the specific areas where available water data on sodium, hardness, TDS, and trace metals exists. Comparisons of areas using polluted and clean water sources should have priorities. There is a need to coordinate an epidemiology assessment strategy to insure that kinds and amounts of pollution and possible health reactions are measured uniformly. A small interdisciplinary group with international support could be convened to develop and design such a strategy.

A city planning overt reuse of wastewater will require many different types of data, accurately collected and analyzed, to determine whether reuse has an adverse effect. Properly designed registries of the incidence and prevalence of chronic disease are efficient basic tools for this purpose. The registry should include data for two or more years before overt reuse, and for several years after reuse, on chronic as well as acute episodes of disease.

Associations of cadmium ingestion and hypertension have been reported as well as an association of hardness with low rates of cardiovascular disease. The substances responsible for this association have not been clearly determined. Some of the contaminants in reused wastewater may be relevant to this association. Comparison studies in like communities with different degrees of covert reuse, or in communities changing their water supply, can assist in resolving these questions.

Rates of the incidence of "serious" chronic disease vary from place to place. Areas exhibiting marked differences in chronic disease rates should be utilized in determining the possible relationships between incidence and concentration of water constituents indigenous to each area. Hospital discharge reports and death certificates can be used to document incidence of "serious" disease.

4. Health Effects/Organics - Many existing water supplies contain appreciable quantities of organic compounds. These compounds may present a health hazard if they are carried into potable water supplies. Efforts are underway to evaluate the toxicity of organic compounds in water supplies where various degrees of covert reuse occur. There is a need to carry out short-term toxicological studies on actual wastewater effluents (and concentrates of them) produced by advanced waste treatment facilities designed for overt potable reuse. Appropriate standard techniques for concentrating refractory organics that maintain the integrity of the chemical components are a prerequisite for these studies. Only chemicals identified from highly toxic fractions and suspected of being chronically toxic should be subjected to longer term testing. These long term tests should include consideration of the synergistic potential.

The practice of disinfection of water for biological safety has brought with it the creation of many new and potentially toxic chemicals. Attention must be paid to the reaction products resulting from the different means of water disinfection, such as the use of ozone, chlorine, and UV light. Residual compounds such as epoxides, aldehydes, and acids need careful assessment.

Research on analytical methodology designed to identify specific organic pollutants, some of which are hazardous to health, should be coordinated with ongoing work in program areas of municipal wastewater, industrial wastes, and drinking water research. Techniques of isolation, concentration, identification, and quantification of organics should be improved. Quantitative recovery methods that do not allow introduction of artifacts as well as more studies on the identification of nonvolatile, higher molecular weight compounds are required.

The recent development of in-vitro bacterial systems serve as sensitive indicators of a candidate compound's carcinogenic/mutagenic potential and should be given serious consideration as a screening tool early in the evaluation of the health hazards of a compound or water concentrate.

Utilization of short term in-vitro mammalian procedures, such as the use of animal or human liver tissue may provide guidelines in the early screening evaluation of organics. The metabolism of the organics found in reused wastewater should be studied to determine the best possible models for toxicity testing.

Relatively simple and rapid gross chemical or biological methods should be developed that give good correlation with the presence of compounds of health significance. These methods must be able to be used for routine monitoring of potable water supplies to give assurance that organics in the water do not constitute a hazard to health. Analyses such as total organic carbon (TOC), chemical oxygen demand (COD), carbon chloroform extract (CCE), volatile organic analyses (VOA), mutagenic microbial assay, and other potential gross analyses should be compared with measured concentrations of compounds found in water that are determined to be potentially hazardous, so that safe levels based on the gross techniques can be established.

The interaction of pollutants should be assessed. The importance of synergistic effects of, for instance, specific chlorinated hydrocarbons with alcohols and their oxidation products (aldehydes and ketones) in amplifying toxicity effects requires additional study.

In order to evaluate the potential health hazards of reclaimed waters for potable reuse, there is a primary need to establish the kinds of organics which are present and their concentrations. Such background information will aid in the formulation of toxicological and other health hazard assessment plans related to organics in drinking water. In addition, these studies will be very useful in comparing characteristics of organics in reclaimed wastewaters with present drinking water supplies.

Epidemiologic surveys of population groups exposed to heavily polluted surface waters are needed. The aim is to determine whether there are any detrimental health effects resulting from such long term exposure. The studies would include testing of the body fluids and tissues for the accumulation of compounds of potential toxicologic importance. The sources of organic pollutants in water are diverse and include municipal wastewater, airborne wastes, industrial and agricultural wastes, and other products of commerce. For certain pollutants, studies should be undertaken to evaluate the relative hazard of these compounds in water as compared to their hazard from other sources.

Because of the diversified problems involved in evaluating the health effects of potable reuse, there is a need to develop a viable and visible program coordinator within EPA to assess the potability of reused wastewater. Also, there is a requirement for a scientific definition of what constitutes "potable quality." When this has been done, the requirement for health effects testing of water from both overt and covert reuse situations will be clarified.

An assessment of hazards to operators of wastewater renovation and reuse facilities should be conducted. Besides the hazards of exposure of workers to chemicals related to the chlorination process, hazards of exposure to ozone or UV light should be considered.

There is a need to coordinate research efforts in the direction of standardizing methods for evaluating the health effects of organics in reused waters to be used for potable purposes. Coordination of research should be undertaken on an international level with such agencies as WHO and the agencies of specific countries directly involved in such endeavors. The output from the various international programs should be integrated and disseminated by periodic newsletters on a regular basis to all participants. The newsletters would address research in progress or planned, tabulation of specific organic compounds identified in waters, and any toxicity data and supporting information.

5. Health Effects/Biological - One of the primary public health considerations for the potable reuse of wastewater is the prevention of communicable diseases by virus and other pathogenic microorganisms. The disinfection process is a major means by which virus are rendered non-infective in any reuse system. In order to achieve disinfection the inactivating agent must reach the virus.

Methods are needed for the determination of the extent to which kill efficiency is affected by adsorption of the organism to, or entrapment in, solids for waters of low turbidity levels (0-1 JTU). Procedures for detecting viruses on or in solids and the means of exposing adsorbed virus to the disinfectant should be developed. Hepatitis A virus is the major enteric virus involved in water borne disease outbreaks, and the development of methods for its detection and identification are of the highest importance. Methods are also needed for the detection, isolation, and assay of other viruses which are excreted from the human enteric tract and which are of public health importance in renovated waters. Information is needed on the mechanisms of inactivation of viruses by disinfection and adsorption. Knowledge of the exact mechanisms would lead to modifications of existing processes to improve their efficiency. The fate of viruses in the treatment system will become predictable when disinfection and adsorption mechanisms are better understood.

There is an unquestioned need for the development of a rapid and simple testing method that would indicate the presence of viruses and other pathogenic organisms in renovated waters. A comparative methods study and evaluation of various cell culture systems should be undertaken with respect to their ability to detect viruses. The best systems should be standardized so that a uniform method would become available to all workers, and would also permit a comparison of laboratory findings between laboratories. Research should be supported to provide a standardization of cell systems that would afford a reproducible method for recovering a test seed of virus and for assaying infectivity titers.

Data are required on the occurrence, survival, and fate of viral and bacterial pathogens in all types of treatment plant sludges, and on the interaction between pathogens in the sludges and soils of varying composition.

There is a need to prepare a state-of-the-art document concerned with the removal and inactivation of microbial pathogens by pH extremes and by chemical coagulation processes. Such information is necessary for a rational approach to the development and standardization of optimum treatment conditions for the removal and inactivation of microbial pathogens.

Information is needed on the minimal amount of ingested virus that will produce infection. This information would provide the basis for establishing viral standards for renovated waters.

A study should be undertaken to determine the extent to which bacteria that colonize activated carbon and other solid contact systems release toxic and pyrogenic materials. The released toxins and pyrogens should be identified and their potential health hazards determined.

There is a need to conduct in-depth surveillance and monitoring for the presence and concentration of viruses in effluents from existing pilot and full-scale wastewater reclamation plants which employ treatment trains that are likely to be used in future production of potable water. Also, data are required on the potential public health hazard of airborne dissemination of pathogenic bacteria and virus. The information would aid in the public decisions related to plant-siting.

6. Socio-Economic Aspects - Attempts to institute potable reuse have not been frequent enough to develop a clear picture of social reaction. Resistance is expected unless clearly defined standards are established that have the support of the public health officials at all government levels.

A national survey, as well as local investigations of areas of potable reuse need and feasibility should be implemented. This includes the identification of the extent to which the U.S. population is presently being supplied former wastewater as part of their raw water supply. A careful analysis of the amount of wastewater in present drinking water sources should be useful in demonstrating the possible degree of pollution of existing water supplies. Public education on current indirect potable reuse is needed. Methods of educating the public, most of whom are using some wastewater in their domestic supply, are required so that they can be informed of this in such a way that they will understand and accept it.

Recycling of municipal wastewater has the potential of impacting the quantities available for upstream and downstream water users. Individual state water laws must be interpreted, modified, or both, to accomodate the implementation of reuse. Identification of significant legal precedents which could constrain the implementation of such reuse is needed.

Numerous regulatory entities exist which may have jurisdiction over various water-consuming activities within a given area. Identification of such agencies and delineation of their respective responsibilities would alert water purveyors to standards, regulations, and legislation that could constrain potable reuse. The environmental assessment could lead to considerable litigation if the appropriate standards, regulations, and legislation are not adequately addressed.

Basic economic data are needed on the value of a certain level of water quality to the consumer. That is, how much money is the average household consumer willing to pay for a higher level of water quality? Planning for reuse would be significantly advanced if data were available on the perceived value of a higher quality water over another quality. This is particularly germane to decisions to implement recycling now, wait until technology is farther advanced, or to develop more expensive conventional supplies.

A study should be conducted on the effects of various methods of encouragement that can be exerted by funding and regulatory agencies on separate water supply and wastewater treatment entities to get them to coordinate their efforts in water management programs that include reuse.

Efforts should be made to determine and forecast the relative importance of the types of sociological, institutional, and economic problems that will be faced by cities pioneering potable wastewater reuse. This would include a determination under varying sets of circumstances, of the proportion of reclaimed water the public will accept in their domestic water supply. There is considerable evidence that public attitudes currently oppose drinking reused wastewater. This attitude persists despite objective evidence that "accidental" or indirect reuse, equal in contamination to direct reuse, is common. The problem faced by a pioneering city is that public attention by tourists, convention-goers, and other non-residents may lead to a negative community reaction.

A number of cities are proceeding to develop water reuse technology on the assumption that potable reuse will become feasible in another 10 to 15 years. The problem which could arise is that public health concerns may not be overcome when scheduled. The possiblity exists that a legal injunction may be granted, on the request of health experts or even environmental groups, which bars implementation of the project. For this reason, assessment of the social and economic impacts of unexpected delays in scheduled reuse become necessary.

History shows that as innovations increasingly become adopted, resistance to adoption fades. Data is needed on the likelihood and extent to which sociological, institutional, and economic problems faced by cities practicing potable use of wastewater will diminish, as more and more cities adopt this practice.

In view of the fact that our present drinking water supplies are often more economical than potable reuse water, it is imperative from an economic standpoint that existing supplies be conserved to the greatest extent possible within the present "life style" of the community. Due to the "unlimited supply" attitude that has existed, the American public has unconsciously become very wasteful of water. A survey directed to a review and compilation of the various water conservation programs that have been instituted by water agencies is needed to disseminate the information and expand the practice of conservation of resources. This would lead to the development of a viable and operable water conservation program for use by utilities.

SUMMARY

It is anticipated that a program undertaking the research previously described will require a minimum of 10 to 15 years of intensive work to develop sufficient information to clearly define meaningful standards that can be applied with confidence to potable waters derived from a polluted source. These standards will have to be based on realistic public health considerations and have the support of public health officials at all government levels.

The goals of a direct or overt potable reuse program are similar to those of the present EPA Health Effects and Water Supply programs which have recently identified organic materials having potential health hazards in many of our Nation's drinking waters. Some of these supplies contain appreciable quantities of wastewaters, and their use for domestic purposes can be classified as an indirect or covert form of potable reuse.

There was a general consensus of the Workshop attendees that the research identified must be addressed by both water supply and wastewater organizations. Even if direct reuse is not implemented, all the same questions which have been raised must be answered, and the technology must be developed to remove potential health hazard constituents present in our water supplies.

Any program of the magnitude required to alleviate the health concerns of both overt and covert potable reuse is not a local or even a national undertaking. International coordination is necessary since other nations such as South Africa, Israel, and some in the European community are facing deteriorating and unreliable water supplies, and are actively researching the problems involved with covert and overt potable reuse.

The research identified at the Workshop will be used to lay out a long range approach which will utilize the results from the ongoing EPA Wastewater, Health Effects, and Water Supply Programs. By clearly defining a strategy for potable reuse the results from these programs can be used as "stepping stones" for a future potable reuse research program.

REFERENCES

1. "Water Policies for the Future," National Water Commission, Washington, D.C., (June 1973).

2. "Statewide Standards for the Safe Direct Use of Reclaimed Wastewater for Irrigation and Recreational Impoundments," State of California Department of Public Health, Berkeley, CA (May 1968).

AGENDA

WASHINGTON, D. C.

WATERSIDE MALL - ROOM 3305

Tuesday, October 28

9:00 am	F. Green - Welcome J. T. Rhett - Conference Opening Dr. T. Kubo - Response Dr. W. K. Talley - Remarks, Office of Research and Development R. A. Canham - Greetings from Water Pollution Control Federation J. T. Rhett - Introduction of Delegates and Confirmation of Agenda
9:30 am	W. S. Groszyk - Planning for Urban Runoff Control Under a Comprehensive Water Quality Management System
10:15 am	Break
10:30 am	R. B. Schaffer - Control of Water Pollution Through Issuance of Discharge Permits - Implementation of P.L. 92-500
11:15 am	E. P. Hall - The EPA Pretreatment Program for Industrial Wastes
12:00 noon	Luncheon - Hosted by the Water Pollution Control Federation
1:30 pm	C. C. Taylor - Municipal Sewer Utility Financing Under P.L. 92-500
2:15 pm	J. T. Rhett - A Perspective on Municipal Pollution Control - The Construction Grants Program and P.L. 92-500
3:00 pm	Break
3:15 pm	J. G. Moore, Jr. - Commission Charge - Tentative Staff Issues and Findings National Commission on Water Quality
4:00 pm	J. T. Rhett - Closing and Announcements

PLANNING FOR URBAN RUNOFF CONTROL UNDER COMPREHENSIVE WATER QUALITY MANAGEMENT SYSTEM

Walter S. Groszyk
Deputy Director, Water Planning Division
Environmental Protection Agency
East Tower, Room 815 - WH-554
401 M Street, S.W.
Washington, D.C. 20460

ABSTRACT

This paper summarizes the approach being developed by the U. S. Environmental Protection Agency for the abatement of urban storm runoff. The approach features the development, at the local or regional level, of specific management practices to control runoff. These practices emphasize non-capital intensive methods of minimizing the pollutant loading of runoff and are formulated through a comprehensive areawide planning effort that assesses the dimensions of the runoff problem, the water quality effects, and abatement needs.

TEXT

The United States Environmental Protection Agency through the States and cities conducted a survey of the estimated construction costs to abate storm sewer pollution. This survey of needs amounted to nearly $250 billion in current dollars. While the number does not generally reflect any engineering plans or detailed surveys, its rough magnitude is staggering and beyond the digestive capacity of the Federal budget. There is simply no foreseeable way that the Federal Government would be able to finance a construction program of this size for this problem. It exceeds by tenfold the total program of landing a man on the moon, and is nearly ten times the total annual contract construction value in the U.S. gross national product for 1973. As an additional perspective, the estimated costs, in this same Needs Survey, for constructing treatment plants, interceptor and collection sewers, and controlling combined sewers totalled approximately $100 billion, with the total need becoming nearly $350 billion.

Areawide Planning

Under the Federal Water Pollution Control Act, an areawide planning program is to be conducted in urban-industrial areas with significant water quality problems. This planning program has just begun, and at the present time planning is under way in 149 areas at a cost of $163 million. This planning covers many of the largest cities of the United States, including New York, Philadelphia, Chicago, and Detroit. These plans will eventually cover the entire United States. Presently, about 45% of the population and 11% of the land area of the United States are covered by areawide planning.

Areawide planning is also called 208 planning, after the number of the section of the Federal Water Pollution Control Act which authorizes it. Areawide planning is quite unique for three reasons. It is expected to be the largest planning program ever funded by the Federal Government; all the financing for the program comes from the Federal Government; and the law requires that the plans be implemented.

Areawide planning conducted for a local area is a comprehensive plan. The plan covers both point and nonpoint sources of pollution. It includes: initial facilities planning for municipal sewage treatment works; an identification of industrial pollution control requirements; and the identification of practicable methods and procedures for the control of nonpoint sources of pollution, including runoff from agricultural, silvicultural, mining, and construction activities.

The planning effort is conducted at the city and county level, and is under the direction of the chief elected officials of local government within the planning area. This will assure that plan development reflects the views of the government officials who will operationally implement the plan. The initial plan is to be completed not later than three years after the planning agency has been approved by EPA. At the time of completion, EPA is required to approve the designation of the management agencies who will carry out the plan. The planning agency continues to function and develops revisions to the plan as they are required.

This stress on locally originated planning conducted under the direction of locally elected officials reflects an appreciation that levels of abatement more stringent than those required by nationally applicable effluent guidelines for industry or uniform secondary treatment for municipalities can often best be addressed by examining the specific pollution problems in that area which remain after national levels of control are applied; assessing the institutional and financial capabilities that exist or can be developed to abate the sources of this remaining pollution; and then making the tradeoff's between alternative control methods to develop the most effective and reasonable way of reaching the water quality objectives.

Best Management Practices (BMP's)

The Environmental Protection Agency, to assist planning agencies in making these tradeoff's, is developing informational guidelines outlining different methods and procedures that can be used to abate nonpoint source pollution including urban runoff. These informational guidelines are called Best Management Practices or BMP's. A BMP is not an "end of the pipe" technological control level, but rather is a way of doing business. It is directed to the activity being conducted; as an example, it may indicate that sediment runoff from a farmer's field can be reduced by changing the manner in which the farmer plows the field. BMP's are being developed for all categories of nonpoint source pollution, including urban runoff.

BMP's for a particular category present an array of alternative practices with varying economic costs and efficiency rates. The practices often identify climatological or topographical suitability. From this inventory of BMP's we expect the planning agencies to select those practices which most fit the abatement needs of their area. The planning agency is not required to use any of the BMP's but may develop an equivalent BMP on its own. While Best Management Practices are generally concerned with non-capital intensive methods of control, they are not exclusively so. A planning agency may also, after analysis, determine that the most effective and economic way to achieve water quality goals for that area is through the construction of facilities and structures.

BMP's and Urban Runoff

Our present perception is that it is neither necessary nor possible to treat all storm waters. A receiving water is subject to stresses caused in part by various natural and uncontrollable occurrences. Many streams experience difficulty during the low flow and high temperature period of later summer. Wet weather conditions represent yet another period of stress.

The true extent of the storm water problem is largely unknown and the lack of any extensive historical studies or concern makes it difficult to characterize.

Considering the area and route that urban runoff takes, it is not surprising that this runoff contains substantial amounts of organic material, inorganic material, inorganic solids, nutrients, heavy metals and micro-organisms. The impacts from this runoff are often increased oxygen demand, high turbidity, and increased eutrophication rates. Additionally, the impact of heavy metals on the aquatic environment has to be considered.

The total pollutant load in stormwater, during storm runoff periods, can be greater than the pollutant load discharged from municipal treatment plants during dry weather. This could preclude meeting water quality standards regardless of the degrees or types of treatment afforded dry weather wastewater flows.

Related problems resulting from unregulated, or poorly regulated, runoff are accelerated erosion of land area and stream banks, sedimentation of channels, increased flooding, increased potential for public health problems and deterioration of aesthetic quality.

Assessing the impact of stormwater runoff is not easy. Part of the difficulty lies in the variability of stormwater runoff. The quantity and quality of storm overflows, for example, can vary with respect to storm characteristics, antecedent conditions, time, location, degree of urbanization and other factors. This variability, the differing problems in new and expanding urban areas compared with existing areas, and the scarcity of information concerning stormwater impact on receiving waters pose challenges to the formulation and administration of an effective management program.

Some of the examples of these varying factors are that: loading rates are lowest in commerical areas; BOD_5 and COD concentrations are lowest in residential and heavy industrial areas, while the COD concentration is highest in commerical areas; cadmium concentrations are relatively uniform across all areas; chromium, nickel, and copper are lowest in residential areas, with lead concentrations lowest in heavy industry areas; and finally, and surprisingly, there is no significant difference between land use category and fecal coliform count.

From the analysis of the specific problem parameters with respect to water quality and with a correlation as to the likely land use areas and the sources of the problem, the planning agency can then analyze which BMP's it might apply to control the problem.

BMP's within an urban/suburban area include source regulation, collection system control, treatment, and an integrated approach using all three. Source control is defined as those measures for preventing or reducing stormwater pollution that utilize management techniques (e.g., good housekeeping methods) and stormwater detention within the urban drainage basin before runoff enters the sewerage system. Collection system control includes all alternatives pertaining to collection system management, such as use of sewers as detention facilities. Treatment, including storage, is another technique. The term storage refers to stormwater being retained for the purpose of treatment as opposed to storage used in source control to attenuate the rate of runoff. Flow attenuation is concerned directly with runoff as it moves over the surface of the urban area; i.e., the initial collection system. Flow attenuation, in an hydrologic sense, means to increase the time of concentration and decrease the magnitude of the peak runoff. In terms of water quality this means that runoff velocities are reduced and less pollutants are entrained. Also, less erosion results because reduced runoff velocity reduces the erosion force. Moreover, large volumes of water are not allowed to rapidly accumulate at constrictions, but flow at reduced rates over a longer period of time, thus reducing the possibility of localized flooding. An integrated approach might include source control to help reduce pollutant loads and runoff rates; collection system control (sewerage) to reduce infiltration and to attenuate the runoff; and treatment as a final stop where required to meet water quality objectives.

The management goals become:

1. Prevention and/or reduction of pollution.

2. Detention or retention of runoff.

3. Treatment of runoff.

An additional goal that should not be overlooked is reuse of stormwater runoff. Reuse of stormwater places urban runoff in the resources category. It should be considered in those areas that can benefit from groundwater recharge and supplemental supplies for both potable and nonpotable use.

The goal of the planning approach is to provide sufficient pollutant reduction to meet water quality objectives at a minimum cost. BMP's for urban runoff should stress source and collection system management, and reuse where applicable. Treatment should be resorted to only when all other lower cost methods have failed to provide sufficient pollutant reduction.

Urban runoff management should initially emphasize the new urban areas. These new areas include land that is in the process of becoming urbanized. These are areas that allow for the greatest degree of flexibility of approach in addressing the long-term problems. At a minimum, urban runoff pollution must be contained to present problems. Once contained, then emphasis can shift to problems associated with existing areas. Of course, for this approach to be effective, complete area coverage is required and the approach should be implemented as appropriate throughout the entire planning area.

For a BMP to be considered as a best management practice, it must meet certain general and specific criteria. Factors that need to be addressed within a BMP include, but are not limited to, the following.

A BMP must be compatible with the hydrology and meteorology of the planning area. The frequency, intensity, duration, and surface area extent of precipitation must be addressed; also, infiltration rates, depression storage, and runoff rates. Groundwater must be considered in relation to recharge areas and levels, and effect on stream channels fed from groundwater.

Runoff from snowmelt in some areas of the country (for example, in parts of the West) produces the major portion of the annual runoff. An important factor in considering snowmelt is the temperature. Other factors to be addressed are wind and humidity.

Topography, of course, is a factor that must be considered. A BMP must be compatible with the slope, length of basin, and type of surface cover of the planning areas.

Geology is another factor to be addressed. Soil types vary widely across the country. A BMP must consider and be compatible with this variable.

The specific examples of BMP's that can be examined are to be considered as being site-specific and are not to be construed as being applicable nationwide.

Source Control

Some examples of source control are:

1. Street sweeping or control through housekeeping.
2. Sewer flushing to reduce first flush effects.
3. Detention basins.
4. Rooftop storage and parking lot storage.
5. Porous paving, to increase infiltration.

Collection System Control

Some examples of collection system control are:

1. Use of existing sewerage as detention facilities.
2. Use of swirl concentrators.

Treatment

Two examples which have been studied and have been found feasible for stormwater treatment are:

1. Micro-straining with air floatation.
2. Contact stabilization.

Institutionally, we expect the planning process will work in the following manner.

The planning agency staff will assess the magnitude and extent of the urban pollution problems. These will be presented to the public and any advisory groups to ensure that a basic understanding of the problems is shared by all.

The staff will then formulate water quality goals based on protecting beneficial uses of water. These will be discussed with the public and will be presented to the advisory committee. The advisory committee overseeing the development of the plan will select the approach which best meets the goals.

Proposed criteria for controlling urban runoff pollution will be prepared by the staff in consultation with the public. An inventory of alternative BMP's which meet the proposed criteria will be made.

The staff will evaluate and make initial selection of those BMP's which meet the water quality objectives, and the public and advisory group will decide which of the BMP's are institutionally and economically acceptable. Final selection of the BMP's will be made then by the staff.

After the urban runoff problem has been assessed, runoff reductions to help meet target load allocations achieved by the use of BMP's often need to be translated into ordinances or regulations.

Within EPA, we believe the use of the areawide planning process together with the systematized assessment of BMP's offers an attractive alternative to total reliance on capital facilities for control of urban runoff. The planning process has just begun, and we would like to report at a future conference on the results from this effort.

CONTROL OF WATER POLLUTION THROUGH ISSUANCE OF DISCHARGE PERMITS, IMPLEMENTATION OF P.L. 92-500

R. B. Schaffer
Director, Permits Division
Office of Enforcement
U.S. Environmental Protection Agency
Washington, D. C. 20460

OBJECTIVES OF THE NPDES PROGRAM

On October 18, 1972, the Amendments to the Federal Water Pollution Control Act established the National Pollutant Discharge Elimination System (NPDES). With the enactment of this new legislation Congress has stated that it is the National goal that the discharge of pollutants into navigable waters be eliminated by 1985. As an interim goal it is stated that there be attained by July 1, 1983, water quality which provides for the production and propagation of fish, shellfish and wildlife and provides for the recreation in and on the water.

Any permit issued under the National Permit System will impose on a discharger of pollutants from a point source certain requirements designed to attain the goals of the Act. Every discharger must make application for a permit and in so doing, provide the permitting authority with data on the discharge. Each issued permit will meet effluent limitations, water quality standards, new source performance standards for new plants, and toxic pollutant standards. Facilities discharging into a municipal waste treatment facility do not require a discharge permit, but the discharger must comply with pretreatment standards promulgated under the Act. Permits will require the discharger to monitor the discharge, to keep records of monitoring activities and report periodically on what is occurring with regard to the discharge.

THE EFFLUENT LIMITATIONS

The new Act provides for uniform effluent limitations for industrial categories and achievement dates. Congress set two interim dates of July 1, 1977 and July 1, 1983, by which different levels of treatment are to be reached. It is a timetable based on advances in technology.

For all discharges other than publicly owned treatment works, not later than July 1, 1977, effluent limitations are to be achieved which represent the application of the "Best Practicable Control Technology Currently Available." At the same time, all publicly owned waste treatment facilities must utilize "secondary treatment" and, if an industrial discharger sends its waste through a publicly owned treatment works, certain "pretreatment standards" must be met. An additional requirement is that by the July 1977 date, effluent limitations may be imposed so that any state law will be met. Not later than July 1, 1983, effluent requirements must be met which represent the "Best Available Technology Economically Achieveable" and, for publicly owned waste treatment facilities, which represent the application of the "Best Practicable Waste Treatment Technology." Any other applicable pretreatment standards must also be attained by that date. Special standards of toxic substances must also be observed for both the 1977 and 1983 targets.

The target dates are 1977 and 1983; they are the outside limits for compliance. The Act envisions that in meeting effluent limitations there will be stages of compliance including attainment of levels of substantial improvement even before these dates. Therefore, most permits will impose a schedule of remedial measures. This schedule will appear as a condition set out in an NPDES permit.

The Agency has requested authority to extend the 1977 date on a case-by-case basis for publicly owned treatment works. However, we do not feel it is necessary to extend the date for other dischargers nor do we expect the National Commission on Water Quality to recommend it to Congress.

BEST PRACTICABLE CONTROL TECHNOLOGY AND BEST AVAILABLE TECHNOLOGY

The Act charges the Administrator with the task for publishing regulations providing "Guidelines" for effluent limitations for point sources after he has consulted with the appropriate Federal and State agencies and other interested persons. These effluent limitations are the ones which shall require the application of the Best Practicable Technology by 1977, and Best Available Technology Economically Achieveable for the 1983 target dates. Two things will be identified in the regulations.

First, they will give meaning to the terms "Best Practicable" and "Best Available" when applied to various categories of industries. In defining "Best Practicable" and "Best Available" for a particular category, such factors as the age of the equipment and facilities involved, the process employed, the engineering aspects of the application of control techniques, process changes, and non-water quality environmental impact (including energy requirements) will be taken into account. In assessing "Best Practicable Control, a balancing test between total cost and effluent reduction benefits is to be made. Cost is also a factor in determining "Best Available." "Best Available" technology is the highest degree of technology that has been demonstrated as capable of being designed for plant scale operation, so that costs for this treatment may be much higher than for treatment by "Best Practicable" technology. Yet economic feasibility will also be a factor in interpreting "Best Available" treatment. Cost effectiveness for either standard is to be confined to consideration of classes or categories of point sources and will not be applied to an individual point source within a category or class.

Second, having interpreted "Best Practicable" and "Best Available" guidelines will be published which will determine what "Effluent Limitations" are to be imposed on dischargers. In these guidelines the degree of effluent reduction attainable through the application of the "Best Practicable Control" and "Best Available Technology" in terms of amounts of constituents per unit of production. These guidelines can then be applied in setting specific effluent limitations on dischargers.

The Agency will promulgate these various standards and guidelines for some 200 classes and categories of dischargers.

TOXIC POLLUTANT EFFLUENT STANDARDS

The Act requires the establishment of effluent standards or prohibitions controlling toxic pollutants. Toxic pollutants are defined as those pollutants, or combinations of pollutants which, after discharge and upon exposure to any organism either directly or indirectly, will "on the basis of information "available" cause death, disease, or other abnormalities in the organism or its offspring. The drafters of the Act had in mine certain substances such as mercury, beryllium, arsenic, cadmium pesticides, etc.

A list of toxic pollutants has been proposed. Effluent standards for those toxic pollutants listed will be published later.

NEW SOURCE PERFORMANCE STANDARDS

Most new plants will be subject to national standards for performance. EPA is to publish a list of categories of sources which must include 27 major types of industries and then issue regulations establishing Federal standards of performance for the new sources within such categories. These standards are to assure that new stationary sources of water pollution are designed, built, equipped, and operated to minimize the discharge of pollutants. The standards are to reflect the greatest degree of effluent reduction which the Administrator determines to be achievable through application of the best available demonstrated control technology, processes, operating methods, or other alternatives. "Best Available Demonstrated Technology" has been described as those plant processes and control technologies which, at the pilot plant or semiworks level, have demonstrated that both technologically and economically they justify use in new production facilities.

At the same time EPA promulgates new performance standards, it is to provide pretreatment standards for newly constructed point sources discharging into public treatment facilities.

WATER QUALITY STANDARDS

The new Act does not ignore the concept of water quality standards in 1977 and 1983 achievements. Water quality standards which were adopted and enforced under the old Federal Water Pollution Control Act (FWPCA) for interstate waters are continued in effect and can be updated and the new ones are to be established for intrastate water bodies where not previously adopted by the States. If water quality standards cannot be protected by the application of best practicable control technology for industries and secondary treatment for municipal wastes before 1977, then more stringent effluent limitations are to be imposed which will pro-tect water quality for public water supplies, agricultural and industrial uses, assure protection of a population of fish and wildlife, and allow recreational activities.

EFFLUENT LIMITATIONS

The permit will contain one or more sets of numerical limitations which must be met by a date specified in an associated compliance schedule. In general, the effluent limitations, with the exception of pH, will be expressed in terms of total weight (lbs/day or kg/day). The effluent limitations in the permit are described in terms of daily average and daily maximum values. The limitations expressed in the permit are based on promulgated effluent guidelines, interim guidance or water quality standards if more stringent limits are necessary to protect water quality. The limitations or standards established by the Agency are to be applied in a uniform manner throughout the country. The standards are minimum technological requirements to be applied even though the receiving water may not require that level of abatement to achieve the desired water quality.

COMPLIANCE SCHEDULE

The compliance schedule will specify when final effluent limits must be attained and may also contain dates for achieving certain plateaus such as development of engineering reports, final plans, beginning of construction, completion of construction and the operation of facilities. Interim dates and requirements are to be specified in the permit as a means of monitoring progress and minimizing slippage. Following each interim date, the permitee must submit a written notice of compliance or non-compliance with the interim requirements. The reports specified in the permit are very important and should be submitted on time. Failure to report, especially on construction progress or compliance, will result in response from the Agency.

MONITORING AND REPORTING

The self-monitoring requirements contained in the permit will be developed on an individual basis with consideration given for the type of treatment, the impact of the proposed treatment facility on the receiving water and the parameter to be measured. The purpose of the monitoring program is to establish that a treatment facility is consistently meeting the effluent limitations imposed in the permit. Data must be recorded and retained on file by the permittee for at least three years. The reporting frequency of monitoring results will be specified in the permit. A uniform reporting form has been developed and will be provided to the permittee. The self-monitoring may vary from State to State as individual conditions are developed to insure compliance with State requirements.

The permits are issued for fixed terms. The maximum duration of a permit will be five years. The majority of permits have been written for that period since it will involve commitment to a long term abatement program. Permits may be written for a shorter period, however, e.g., the State may require it or the facility may cease operation.

STATE CERTIFICATION

After drafting, a permit is forwarded to the appropriate States for certification. The State has the right to add additional requirements in monitoring, compliance, and additional or more stringent effluent limitations. The Agency, upon receipt of certification requirements, will place these in the permit. Any challenge to any State certification requirements must be through State administrative procedures.

STATUS OF THE PROGRAM AS OF JUNE 30, 1975

EPA and 24 States which were delegated the authority to issue NPDES permits had, as of June 30, 1975, issued 20,091 Industrial permits; 16,664 Municipal permits; 1,548 Agricultural permits; and 1,988 Federal Facility permits making a total of 40,291 permits issued. Approximately 1600 EPA issued permits have been challenged through Administrative Processes. Of these, 400 have been resolved through discussions between interested parties, e.g., government, industry, and public interest groups. We expect very few appeals to proceed through this process and into our courts.

A study to determine the total amount of certain pollutants that will be removed from our Nation's waters due to the implementation of P.L. 92-500 and the industrial portion of the permit program resulted in an estimated reduction of approximately 12 million pounds per day of BOD and 28 million pounds per day of suspended solids.

The continuation of our effort will now shift into compliance monitoring to assure that the terms and conditions of the permits are met and the goals of the Act achieved.

THE EPA PRETREATMENT PROGRAM FOR INDUSTRIAL WASTES

Ernst P. Hall
U.S.E.P.A.
401 M Street, S.W.,(WH552) Washington, DC 20460

ABSTRACT

The Federal Water Pollution Control Act Amendments of 1972 (PL 92-500) directs the promulgation of Federal standards for pretreatment of industrial waste waters which are introducted into a publicly owned treatment works. These standards will require pretreatment to control pollutants which would interfere with a pass through a treatment works. Enforcement will be primarily at the local level with a Federal overview and presence.

Pretreatment of industrial wastes before introduction into a publicaly treatment works (POTW) has been discussed by Mr. Sutfin at the second U.S./Japan Conference. Since that presentation there have been a number of refinements in our thinking and approach to pretreatment. This paper reflects the present status of these refinements.

The Federal Water Pollution Control Act Amendments of 1972, were designed by Congress to achieve an important objective - to "restore and maintain the chemical, physical, and biological integrity of the Nation's waters." Primary emphasis for attainment of this goal is placed upon technology based regulations. Existing industrial point sources which discharge into navigable waters must achieve limitations based on Best Practicable Control Technology Currently Available (BPT) by July 1, 1977 and Best Available Technology Economically Achievable (BAT) by July 1, 1983 in accordance with sections 301(b) and 304(b). New sources must comply with New Source Performance Standards (NSP) based on Best Available Demonstrated Control Technology (BDT) under section 306. Publicly owned treatment works (POTW) must meet "secondary treatment" by 1977 and best practicable waste treatment technology by 1983 in accordance with sections 301(b), 304(d) and 201 (g)(2)(A). Users of a POTW also fall within the statutory scheme as set out in section 301(b). Such sources must comply with pretreatment standards promulgated pursuant to section 307.

Limitations and standards applicable to direct dischargers are established for categories and subcategories of point sources. This same categorization is applied to pretreatment and pretreatment standards, generaly, will be established for each category or subcategory of industrial point source discharge. A general pretreatment regulation for existing sources (40 CFR 128) was adopted some two years ago and is now undergoing revision. The revised regulation (40 CFR 403) is expected to provide a regulatory basis for both existing and new sources.

The term "pretreatment" means the application of physical, chemical and biological processes to reduce the amount of pollutants in or alter the nature of the pollutant properties in a waste water prior to discharging such waste water into a publicly owned treatment works. The basic purpose of pretreatment is "to prevent the discharge of any pollutant through treatment works...which are publicly owned, which pollutant interferes with, passes through, or otherwise is incompatible with such works." The intent is to require treatment at the point of discharge complementary to the treatment performed by the POTW. Duplication of treatment is not the goal. Pretreatment of pollutants which are not susceptible to treatment in a POTW is absolutely critical to the attainment of the overall objective of the Act, both by protecting the POTW from process upset or other interference, and by preventing discharge of pollutants which would pass through or otherwise be incompatible with such works.

Pretreatment standards should allow the maximum utilization of a POTW for the treatment of industrial pollutants while preventing the misuse of such works as a pass-through device. The standards also should protect the aquatic environment from discharges of inadequately treated or otherwise undesirable materials.

The primary technical strategy for establishing pretreatment standards consists of the following provisos: (1) pretreatment standards should allow materials to be discharged into a POTW when such materials are similar, in all material respects, to municipal sewage which a "normal type" POTW is designed to treat; (2) pretreatment standards should prevent the discharge of materials of such nature and quantity, including slug discharges, that they would mechanically or hydraulically impede the proper functioning of a POTW; (3) pretreatment standards should limit the discharge of materials which, when released in substantial concentrations or amounts, reduce the biological effectiveness of the POTW or achievement of the POTW design performance, but which are treated when released in small or manageable amounts; and (4) pretreatment standards should require the removal, to the limits dictated by technology, of other materials which would pass through -- untreated or inadequately treated -- or otherwise be incompatible with a normal type POTW.

In addition to these provisos, it appears to be administratively necessary and technically desirable to establish a volume cutoff or limit below which most materials may be discharged into a POTW, while requiring pretreatment standards for larger flows and more hazardous materials. This is intended to be accomplished by defining, for the purpose of the regulation, a major contributing industry is a discharger who either (a) has a flow of 50,000 gallons per day, or (b) has a flow equal to or greater than 5% of the capacity of the POTW. Any discharger meeting either of these requirements would be subject to all pretreatment standards while a discharger not classified as a major contributing industry by this criteria may not be required to meet specific numerical pretreatment standards. The specific determination is to be made in each subpart and for some particular subparts it may be desirable to alter or change the definition of a major contributing industry in order more properly to apply pretreatment standards, particularly where use of the volume cut-off would not provide adequate protection to the environment.

The first proviso is clear in its application and materials meeting this proviso should be allowed to be introduced into a POTW without pretreatment. Other applications of these provisos will be discussed in the following paragraphs.

The control of influent pH is usually adjusted adequately, particularly for mildly acid wastes, by the alkalinity and buffering capacity of normal municipal waste waters. Additionally, if necessary, treatment of pH can readily be accomplished by chemical addition in a POTW. However, highly acid wastes characterized by materials having a pH below five have the capability for destroying the sewer pipes and sewage treatment facility itself because of their ability to attack metal, concrete and mortar joints. One particularly adverse reaction from the corrosion of acid wastes is to destroy the integrety of in concrete sewers, thereby allowing the infiltration of water during a rainy season. For this reason, very low pH wastes -- below a pH of 5.0 -- are included as prohibited wastes even though pH is generally considered to be adequately treated in a POTW.

Heat is defined in the Act as a pollutant. In most cases, heat in fairly substantial quantities can be discharged into a municipal sewage system along with waste water without causing an upset or other difficulty in operating the POTW. As a matter of fact, some heat, particularly in cold weather, may prove to be beneficial, and may accelerate the effectiveness of the treatment process. However, the normal POTW includes biological treatment systems whose performance can be affected adversely if an excess of heat is found in the treatment plant itself. This point of damage to biological activity is generally considered to be 40¼C (104¼F). Hence, some safeguard is needed to prevent an excess of heat being discharged to the treatment plant while still allowing lesser amounts of heat to be discharged to and dissipated in a POTW.

Slug discharges which cause an upset in the treatment process and a subsequent loss of treatment effectiveness are undesirable both for environmental and treatment plant operational reasons. Defining a slug discharge in quantative terms is difficult. It is commonly recognized that the peak two hour flow rate of normal municipal sewage is about two times the ratio of the average daily flow. This ratio holds for both hydraulic loading and for oxygen demand (BOD) loading. In order to establish a readily definable discharge level at which an industrial user may become liable for causing a POTW upset, a slug discharge is defined based on the normal maximum to average ratio. However, the prohibited waste section does not prohibit slug discharges per se but only prohibits slug discharges which cause a POTW upset.

Some materials are known to be treated effectively in small concentrations in a POTW but are not treated effectively whenever the amount of such materials exceeds the system's tolerance levels. Regulation of these types of materials can effectively allow the POTW to treat as much of the pollutant as it reasonably can, while preventing an excess of such material from passing through untreated or reducing the treatment effectiveness of the POTW. One such material currently under review by the Agency is oil and grease of a mineral origin. The Agency is considering establishing a general limitation setting forth a specific concentration as a pretreatment standard for this particular parameter and a request for public comment on this proposal has been published in the *Federal Register* (40FR17/62). This general limitation would be implemented in each subpart regulation rather than in a general regulation. Other materials such as ammonia, phenol and cyanide may be considered for limitation in the same manner as oil and grease of a mineral origin.

Materials may at times be introduced into a POTW in industrial waste waters for which no treatment effectiveness data for a normal type POTW are available or for which the known data indicate that treatment effectiveness in the POTW is highly variable or inadequate. In such cases, it is obvious that the POTW cannot be depended upon to effectively and consistently remove the pollutant in question. Under these conditions the Agency expects to consider the application of BPT or NSP limitations as the pretreatment standard for these specific materials. Materials which may be included in this category would include metals such as copper, nickel, chromium, zinc and arsenic, and selected organic materials.

Regulations under sections 301 and 306 generally have been established allowing the discharge of a quantity or mass of pollutant related to a unit of production or other production vector. This basis for limitation has the considerable advantage of reducing the discharge of the amount of pollutants to a finite quantity while encourging conservation in the use of water and the reduction in the generation of waste water within a manufacturing process or operation. The Agency believes that mass limitations best fulfill the purposes of the Act. Mass limitations based on similar considerations appear to be the most sound and effective mechanism for reducing the amount of pollutants discharged to a POTW whenever such pollutants would pass through or otherwise be incompatible with such works. The Agency intends to use this concept of limiting the mass of pollutants discharged as the technical basis for the establishment of pretreatment standards for many pollutants.

The enforcement strategy, which the Agency proposes to employ to achieve pretreatment of industrial wastes envisions the application and enforcement of these pretreatment standards by State and local bodies including the POTW receiving and treating the industrial waste waters. It has been determined that many State and local authorities are not yet able to apply production related mass limitations. Moreover, the Act does not provide for pretreatment permits analogous to the NPDES permits of section 402. For this reason, the Agency, at this time, expects to promulgate pretreatment standards which are based on the discharge of a specified quantity of pollutant related to a production vector (e.g., lbs of pollutant per ton of product), but which are stated as a concentration of pollutant in the discharge from a particular industrial process or unit operation. It is anticipated that the rationale for the derivative of the pollutant concentration standards will be described in detail in the preamble to each pretreatment standard for an industrial subcategory. It is also anticipated that restrictions and constraints against water use and dilution may be included when appropriate for each subpart. It is intended that such pretreatment standards be applied at the individual process unit. Additionally an alternate procedure will be made available, as is appropriate for each subpart, so that mass limitation may be applied if both the industrial user and POTW operator desire.

The Agency believes that the use of pollutant concentrations as a pretreatment standard for those materials which may pass through untreated, or are otherwise incompatible with a POTW is an interim measure made necessary by the practical constraints of enforcement. At some future revision of these pretreatment standards, the Agency anticipates that the concentration numbers will be abandoned and the mass limitation will become the sole pretreatment standard.

All of the pretreatment standards being considered are intended to apply to users of a "normal type" of publicly owned treatment works which is basically designed and intended to treat domestic waste waters to achieve the secondary treatment standards as established in 40 CFR 133 and as required by the Act. The secondary treatment standard requires that a sewage treatment plant, in addition to controlling pH and fecal coliform, reduce the amount of biochemical oxygen demand (BOD5) to 85 percent or less of the influent value or to 30 mg/l in the discharge, whichever is the more stringent. A similar restriction is applied to suspended solids.

There are a number of sewage treatment systems, which when properly designed and operated, meet these requirements on a consistent basis. These include the activated sludge system and its modifications, trickling filters, and stabilization lagoons or oxidation ponds. There are a number of activated sludge system modifications which incorporate variations on the amount of sludge recirculation, the amount of air or oxygen supplied to the reaction chambers, the use of pre- and post-chlorination, and the use of sludge digestion, sludge combustion, or land filling as mechanisms for disposal of the sludge generated. The retention time of sewage in such systems generally is short; it is nominally considered to be 6 hours while retention times as short as 3 or 4 hours are not uncommon. Trickling filters are often used where the input waste water is relatively constant and where savings in power and operator attention are needed. Stabilization lagoons or oxidation ponds can be used where the necessary land area is available and where climatic and soil conditions are such that the long retention times required by such lagoons or ponds can be achieved. "A normal type" POTW should not have regular, substantial chemical additive needs for the purpose of removing materials other than BOD and TSS.

Existing publicly owned treatment works rarely include processes such as physical chemical treatment (which is only now becoming a full scale reality in a few areas) or special variants or combinations of biological treatment units that are primarily intended to address the special needs of industrial waste water pollutants rather than domestic waste or water quality requirements.

Variations from the promulgated pretreatment standards may be necessary in certain circumstances to compensate for factors not adequately considered in establishing these standards. This has been recognized in the establishment of other industrial effluent limitations and is equally applicable to pretreatment standards. Two kinds of variants appear to be appropriate depending on the particular circumstance.

In the preparation of the development document for each point source category all of the information which the Agency could collect concerning processes and procedures related to the industry subcategory was collected and analyzed. It is possible, however, that certain facts did not become available to the Agency and could not be employed in decisions related to the pollutants which may be discharged from a particular industry operation or would be related to the treatability or impact which such pollutants might have upon a POTW. For this reason, a variance clause is provided which would allow the establishment of a pretreatment standard, other than that promulgated in the applicable subpart, in those cases where it could be shown that factors related to the industry category fundamentally different from those considered in the development document exist and that these factors require the establishment of a different pretreatment standard.

An analogous situation may occur with respect to factors related to the publicly owned treatment works. A variance clause is provided to allow a different pretreatment standard to be established for those cases where a publicly owned treatment works can be shown to be substantially different from the normal type of publicly owned treatment works on which pretreatment standards are being based. Some of these types of installations are known to exist or are in the planning or design stage. However, at this time it is difficult to establish a separate regulation which would make an allowance for different factors in such publicly owned treatment works.

Although both EPA and the States will play major roles in enforcing pretreatment requirements, the Agency believes that local governments will probably have to play the most important role in any successful enforcement program. Local governments operate the POTWs, which are a vital part of the overall effort to clean up the nation's waterways, and so are sensitive to and directly affected by the pretreatment program. They are closest to the problem and are already frequently involved in related areas such as regulation of sewers and collection of user charges. Moreover, a local role in pretreatment enforcement is consistent with the partnership of Federal and local effort found in the construction grants program and other parts of the Act.

As those with the most immediate stake in the success of the pretreatment program, both in terms of protection of the proper functioning of the POTW and in terms of protection of the local environment, local governments will be the first line of defense. One way they may exercise their crucial role is by means of a local ordinance - a preferred route, and one specifically preserved by the Act. It is expected that each manager of a treatment works would provide for such standards." Local governments may also use the citizen suit provisions of section 505. Section 505 is available because local governments are "persons" as defined in the Act "having an interest which is or may be adversely affected". The citizen suit provisions allow suit to enforce a Federal or State pretreatment standard either against the industrial user of the POTW or against the State or Federal government (for failure to take proper action). The Agency anticipates that pretreatment guidance published pursuant to section 304(f) will be of assistance to local governments in carrying out their responsiblities.

The Agency believes that parallel efforts of all three levels of government will be needed for a successful pretreatment program. To the maximum extent possible, EPA will encourage and assist State and local enforcement action.

MUNICIPAL SEWER UTILITY FINANCING UNDER PL 92-500*

C. C. Taylor, Program Analyst
Environmental Protection Agency
1421 Peachtree Street, N. E.
Atlanta, Georgia 30309

ABSTRACT

The Water Pollution Control Act Amendments of 1972 imposed significant financial requirements upon grantees under the Federal Construction Grants Program administered by the Environmental Protection Agency. This paper discusses the legislative history and implementation experience of these grant conditions which are commonly referred to as the user charge and industrial cost recovery requirements.

All recipients of Federal construction grants must demonstrate legal, financial, and managerial capability to complete construction and provide adequate operation and maintenance during the life of the facility. Grantees must also develop and implement user charge systems whereby all users pay the costs of operation, maintenance and replacement in proportion to their use of the treatment facility. Such charge systems must also include provisions for reimbursement of Federal construction costs allocable to industrial users.

The Environmental Protection Agency's implementation of these statutory requirements is impacting the institutional pattern of municipal sewer utility management. More adequate operation and maintenance is assured, and greater equitability in the distribution of costs is being attained.

Public response to the imposition of user charges has been reasonably receptive. Compliance with industrial cost recovery requirements continues to generate controversy, particularly with respect to applicability, cost allocation, and accountability.

INTRODUCTION

During the long and somewhat torturous legislative history of The Water Pollution Control Act Amendments of 1972, the Honorable Robert E. Jones of the U. S. House of Representatives characterized this legislation for his collegues as follows: "Mr. Chairman, this is an enormously complex bill, and necessarily so, because our water environment has become enormously complicated because of the urbanization and industrialization of our society. Our legislation must take into account the myriad of the water needs issue" (1).**

Subsequent to enactment, the act has been called many things, ranging from the most significant legislation of the decade to the most comprehensive, the most complex and the most confusing legislation ever enacted at any time or place in the history of man. Whether or not either of these latter characterizations are justified, PL 92-500 is, without doubt, comprehensive in scope. Many of its provisions are complex, and implementation of some of the provisions have been accompanied by some confusion and controversy. This includes implementation of significant provisions relating to cost-sharing and cost allocation. The emphasis of this paper is focused upon these financial aspects of the subject legislation.

---

*Paper prepared for presentation at Fourth U. S./Japan Conference on Sewage Treatment Technology, Washington, D. C., October 28-29,1975.

**Numbers in parentheses designate references on page 6.

BACKGROUND OF PL 92-500

In the spring of 1967, a joint Committee of the American Public Works Association, the American Society of Civil Engineers, and the Water Pollution Control Federation reviewed the most significant problems of the broad area of administrative, legislative, and financial issues of municipal water and sewer service. As a result of the initial review, the Committee decided to focus its activities upon the most urgent area, namely wastewater financing and charges (2). This was indicative of the general conclusion that most of our municipalities were not adequately prepared, at that time, to assume and manage the rapidly escalating financial and administrative responsibilities of sewer service.

A leading management consultant firm, working under contract with EPA Region VIII, found this general situation little changed by 1972 (3). Our nation is, of course, geographically large and widely diverse with respect to political structuring. Obviously, this broad generalization did not apply to all municipalities individually. However, as a general rule, municipal sewer utilities were operated largely as a public service financed by annual appropriations from general revenues. Generally, cost accounting systems, and their attendant legal and financial institutions were not adequate for efficient operation as financially self-sustaining public utilities. It was in this atmosphere that The Congress deliberated legislation which culminated in PL 92-500.

During this legislative process, many complex and controversial issues were discussed and debated, after which some were resolved and some apparently compromised. These deliberations are reflected in the committee and conference report (1).

Of the significant issues debated, two pertain directly to our subject of financial management. First, Congress fully recognized that the costs of attaining the desired levels of clean water were going to be large - we might even say enormous.

The most recent "Needs Survey" cost estimate for the backlog of municipal facilities which are normally funded under the EPA Construction Grant Program - that is, only treatment facilities and attendant interceptors and outfalls - was at the level of approximately 50 billions of dollars (4). We have known for some time that attainment of the desired levels of pollution control was going to be costly, and Congress was fully aware of this as they legislated this act.

Second, the committee reports reflect that Congress also was fully aware of the basic necessity of getting maximum return for each dollar of this enormous investment and that this could be done only with improved and adequate operation and maintenance. Why go to the expense of building these facilities if they were not going to be operated and maintained in such a way that they would do the job for which they were designed?

As a result of these deliberations, the Congress reached some basic decisions. First, the level of Federal cost sharing for the construction costs of the large backlog of needed publicly-owned municipal facilities would be raised to 75 percent.

Second, it would be necessary to find some way to move our municipal sewer systems to a sounder financial basis whereby they could become more financially self-sufficient. This should be done by promoting a shift of the sewer service function from a public service basis to a public utility basis whereby:

a. Wastewater treatment and control service would be paid for by the users of that service.

b. The users would pay these costs on the basis of the extent of their use of the system. In this way, there would be an economic and financial incentive to reduce waste discharge or at least hold it to an amount for which each user would be willing to pay.

Third, after extended, and apparently, heated debate about the Federal funding of publicly-owned facilities which would treat industrial wastes, the Congress reached this basic conclusion. The Federal Construction Grant Program was initially designed to provide Federal funding aid to municipalities for the large backlog of needed facilities. Consequently, it would not be appropriate for this funding program to provide a subsidy to industrial users of municipal facilities. This would be significant economic and financial discrimination against those industries that did not have the opportunity of discharging to municipal, publicly-owned facilities (5).

These conclusions are articulated in Section 204(b)(1) of PL 92-500 in the form of three basic provisions:

1) For all projects funded with Federal construction grants, the grantee must demonstrate legal and financial capability to complete construction of the facilities and to adequately operate and maintain the facilities throughout their useful life.

2) The grantee must agree to develop and put into effect a user charge, sewer service fee system whereby all users of the system will pay a share of the operation, maintenance, and replacement costs in proportion to their use of the system.

3) For all such grant assisted projects that treat industrial waste, the charge system must include provisions for industrial users to pay back to the grantee their proportional share of construction costs, at least to the extent of the Federal grant.

The latter two provisions were essentially new limitations of the Federal Construction Grants Program, and EPA experience with their implementation impinges upon the current pattern of municipal sewer utility management. These are the provisions of PL 92-500 commonly referred to as the user charge and industrial cost recovery grant requirements (6).

EPA EXPERIENCE WITH USER CHARGE AND INDUSTRIAL COST RECOVERY GRANT CONDITIONS

User Charges For OM&R Costs

It is clearly apparent that the primary objective of the statutory requirement for a user charge system is to assure sufficient revenue for adequate operation, maintenance, and replacement of operational components during the useful life of the grant-assisted facility. There has been almost universal acceptance of this objective, in principle. With the exception of some normal resistance to rapidly increasing charges as higher levels of treatment are installed, the users of municipal sewer service have been quick to grasp the logic of maximizing returns from these large capital investments through more efficient operation and maintenance. Likewise, they appear to have readily accepted the necessity of raising sufficient revenue for this objective. There has been far less universal acceptance of the source of revenue and the distribution of these revenues as required by interpretation of the statute.

At the time this legislation was inacted, a significant proportion of our municipalities obtained some or all revenues for provision of sewer services from ad valorem property taxes and other nonuser sources. Many municipalities continue to do this for some components of total costs.

In consideration of Congressional recognition of the diversity of legal and financial factors that existed among jurisdictions, EPA proceeded to draft regulations which would permit reasonable flexibility in the design of user charge systems that would meet the unique requirements of each grantee jurisdiction (5). This interpretation proposed approval of user charge systems including ad valorem tax revenues for OM&R costs if the grantee could demonstrate that revenue was reasonably proportional to sewer use among major classes of users.

The Comptroller General of the United States ruled that such user charge systems would not meet the statutory requirements for proportionality between classes of users nor among users within classes (7).

Therefore the grant condition currently requires that all OM&R revenue, including revenue for costs allocable to non-excessive infiltration/inflow and stormwater treatment, be obtained from user charges.

This categoric requirement continues to be controversial and causes implementation problems particularly where there is a long established custom of relying on property tax revenue and where municipalities find it expedient to continue to do so for funding of local capital costs. The latter practice, of course, necessitates carrying the costs of legal and institutional arrangements incident to the two or more sources of revenue.

As a consequence of this experience, EPA has requested amendment of the statute to permit use of ad valorem based user charge systems under certain qualified conditions. This amendment is now pending before Congress.

Another significantly controversial aspect of the user charge grant condition is the criteria of average-unit pricing with no significant quantity discounts. We have a long historical pattern of public utility rates based upon marginal-unit pricing, affording significant quantity discounts to large-volume users. Such a pattern applied to most public utility services such as electric power, natural gas, water supply, and sewer service.

In order to provide an economic and financial incentive to minimize waste discharge as well as promote the principle of imposing the costs of pollution abatement directly upon the source, the statute was drawn to require that user charges for OM&R be based directly upon factors that significantly affect those costs. Although our agency implementation policy provides for considerable flexibility in the allocation of OM&R costs among parameters of flow and flow characteristics, as well as flexibility in the allocation of costs not directly related to these parameters, our policy requires essentially uniform rates for comparable services among all users.

This grant limitation criteria often accounts for the most significant shifts in relative cost distribution among classes of users, particularly for a shift in relative cost burden from the residential class to the industrial class. Obviously, the extent of change in either the magnitude of costs or the relative distribution of those costs is dependent upon previous levels of service and charges with which current conditions are compared.

EPA regulations require that the grantee implement the user charge system throughout the entire service area to cover OM&R costs on all facilities. We are pleased to learn that this criteria has encountered relatively few problems with the exception of some difficulties in obtaining necessary agreements with recipient communities in regional systems.

Although our agency experience is relatively limited, there seems to be little doubt that operation and maintenance will be materially improved by assurance of more adequate revenues. As the pollution abatement cost burden inevitably increases, our experience would appear to indicate that the attainment of greater equitability is also conducive to accomplishing our major water quality goals.

Industrial Cost Recovery of Federal Construction Costs

As previously indicated, a significant Federal construction grant condition is the requirement for reimbursement of Federal construction costs allocable to industrial users. Although there is a long precedent for such reimbursement of Federal funding in the water resource development field, this feature is entirely new to the Federal grant program for sewer construction.

Immediately subsequent to enactment, there was public criticism from the industrial sector. Industrial users of municipal systems expressed resentment at being the only beneficiaries required to reimburse the grant assistance. This argument was supported by the contention that they were fully comparable contributors to the general revenues from which these grant funds are appropriated. Although this criticism has not disappeared, it has apparently mitigated as industrial dischargers become more aware of the relative costs of using publicly-owned municipal systems as compared with privately-owned facilities.

EPA implementation problems are more closely related to the technical complexities of applicability, cost allocation, and accountability. Perhaps the best indication of the scope of these difficulties is the fact that promulgation of final agency guidelines is just now nearing completion.

It was apparently an objective of Congress to impose this differential reimbursement requirement upon commercial process wastes from the production of consumer products for which pollution abatement costs could be shifted to product prices. In our diverse, but highly integrated and industrialized society, this delineation of applicability has not proved to be easily attained. Such a criteria clearly extends beyond the manufacturing category. EPA regulations define applicability to include A, B, D, E, and I divisions of our Standard Industrial Classification System (8).* This includes the service industry. Exemption from industrial cost recovery is provided for discharges that are primarily segregated domestic wastes from sanitary conveniences. This whole complicated criteria has almost defied practical implementation.

Although the issues of cost allocation are somewhat amenable to technical analyses, such analyses have not yet attained commonly accepted precedents for the parameters of allocating costs nor for the distribution of construction costs among these parameters. Our regulations provide for considerable flexibility, in recognition of the fact that these will vary significantly by type of facility and level of treatment. In the absence of fairly rigid implementation criteria, continuing controversy apparently can be expected as representatives of major user classes compete for financial advantage.

In accord with the statute, the grantee is charged with specific responsibility for collection of the industrial cost recovery revenues. The grantee reverts one-half of this revenue to the U. S. Treasury and retains the remainder. Eighty percent of retained revenues must be used for expansion or replacement of eligible facilities. The eligible project costs for subsequent grants are reduced by the amount of unexpended funds retained for expansion and replacement.

The reaction of grantees to the industrial cost recovery requirement has been less than enthusiastic approval. This is apparently due largely to the rather heavy administrative burden for which they often seem to feel they have no incentive to assume. This introduces what now appears to be a most significant implementation problem for EPA. Although our implementation experience in this area is just beginning, the problems of monitoring and audit accountability are significant. The grant conditions will require multiple accounting procedures for the various revenue accounts. They may also result in multiple cost allocation and amortization procedures for the various components of total costs. These additional requirements impose upon an institutional framework that was inadequate to the task even prior to this imposition.

This brief discussion of some of the more significant implementation problems should be considered tentative and somewhat exploratory. EPA feels that progress is being made and the agency is actively exploring the potential for improvements both within the framework of the existing statute, as well as possible statutory modifications (9)(10). Such considerations are being focused particularly upon the issues of cost sharing and financial management. It is hoped that our experience is of interest to you and may be of some help to you as we pursue our mutual goal of an improved environment.

---

*A-agriculture, forestry and fishing; B-mining; D-manufacturing; E-transportation, communications, electric, gas and sanitary services; and I-services.

REFERENCES

1) A Legislative History of The Water Pollution Control Act Amendments of 1972, Committee Print Serial No. 93-1, Library of Congress, Vol. 1, January 1973, page 356.

2) Financing and Charges For Wastewater Systems, A Joint Committee Report, American Public Works Association, American Society of Civil Engineers, Water Pollution Control Federation, 1969.

3) Financial and Institutional Arrangements For Wastewater Management, Denver SMSA, by Wilbur Smith and Associates, Environmental Protection Agency, April 1973.

4) Cost Estimates For Construction of Publicly-Owned Wastewater Treatment Facilities, 1974 "Needs" Survey, Final Report to the Congress, Environmental Protection Agency, Revised May 6, 1975.

5) Report of The Committee on Public Works United States Senate, Together with Supplemental Views to Accompany S2770, Report No. 92-414, USGPO, Washington, D. C., 1971, pages 28-30.

6) Public Law 92-500, 92nd Congress, S2770, October 18, 1972, Section 204(b)(1).

7) Comptroller General of the United States, Decision File B-166506, July 2, 1974.

8) Standard Industrial Classification Manual 1972, Executive Office of The President, Office of Management and Budget, USGPO.

9) Evaluation of Alternative Methods For Financing Municipal Waste Treatment Works, Socioeconomic Environmental Studies Series, Office of Research and Development, EPA, 600/5-75-001, February 1975.

10) Analysis of Cost Sharing Programs For Pollution Abatement of Municipal Wastewater, Socioeconomic Environmental Studies Series, Office of Research and Development, Environmental Protection Agency 600/5-75-031, November, 1974.

A PERSPECTIVE ON MUNICIPAL POLLUTION CONTROL - THE CONSTRUCTION GRANTS PROGRAM & PL 92-500

John T Rhett
Deputy Assistant Administrator
Office of Water Program Operations
U S Environmental Protection Agency
Washington, D C 20460

ABSTRACT

The Federal Water Pollution Control Act Amendments of 1972 provide for an $18 billion program for construction of municipal wastewater treatment works. Almost $11 billion of that sum remains to be obligated. However, the pace of obligations is rapidly increasing to a goal of $400 million per month. Problem areas encountered in implementing the program in its first three years include the 1977 deadline for secondary treatment, user charge requirements, program management, personnel needs and participation of State agencies. Specific actions to remedy these problems have been taken, including proposing amendments to the law. Several major areas of the program will receive attention in the future including institutional systems, operation and maintenance, development and demonstration of new technology and construction needs and priorities. EPA expects to make recommendations to Congress on future funding of the program.

TEXT

My topic today, the Federal Water Pollution Control Act, as amended in 1972, is a subject that gives me great pleasure to address. I appreciate this opportunity to comment on the strengths and shortcomings of the law, and some current administrative and legislative initiatives as they pertain to the Municipal Water Clean-Up Program.

Since one of our major administrative tasks under the law is to provide funds to get construction of municipal treatment works rapidly underway, I am happy to report first on the status of obligations. As you may know, obligations are Federal funds that are committed as grants to cover 75 percent of the eligible costs of specific municipal wastewater treatment construction projects. And, our EPA grants program is quickly becoming the world's largest single public works program. Almost $11 billion remains from our $18 billion authorization, and we are moving quickly to obligate all of that sum by the statutory deadline, September, 1977.

Currently, our obligations under the new law have reached over $7 billion and projects being completed under the old law have grants of $4.3 billion. Further, our monthly obligation rate in FY 1975 averaged about $186 million over the FY 1974 rate. We went from $115 million to over $300 million, and we confidently expect to raise our average over $100 million more to reach our goal of over $400 million per month for this fiscal year.

I am also pleased to report that the Federal reimbursement program to municipalities is almost completed. This is for construction begun in advance of the Federal grant, and which is considered eligible for a grant. Over $1.6 billion has been obligated and over $1 billion paid out. And, the half a billion dollars in remaining funds are presently being awarded.

Translating investments into projects we have more than 4,000 individual projects funded at 75 percent underway and almost 2,000 additional projects still being completed under the old law. Once finished our new law projects will constitute a $24 billion total investment in public wastewater treatment works, and added to the public wastewater treatment works we have under the former grants program, our national treatment investment would approximate $42 billion by the year 1977

At the present date, we are almost half-way towards our goal of obligating all the grant monies that were authorized under PL 92-500, but the most heartening fact in this report is the rate at which the funding levels have increased this past year. The progressive picture today presents a contrast with the program in earlier days. To appreciate our progress and the initiatives and actions that are currently pending, we must step back in time.

By last year the criticism directed at the program and the law, PL 92-500, had mounted to a very high level. The concerns that were voiced were so general they caused questioning of the program's credibility.

To cite just a few of the major issues, critics attacked:

--The goals of the Act. "They were impossibly idealistic."

--The funding of the Act. "It was inadequate to meet the 'needs'."

--And the deadlines were "unrealistic."

None of the legal mandates anticipated the capital funding requirements nor the local difficulties in raising the matching share of 25 percent. Moreover, as delays lengthened, the costs continued escalating. Construction costs had risen at the rate of 15 percent per annum, and only recently have they shown any signs of abatement.

The secondary treatment deadline, within the Act, and the need to amplify the States' role have also called for legislative change. Since many of the project planning decisions that must be made are based primarily on "best judgment," and the best planning information is located at the State and municipal level, the lack of any real provision for State participation has riveted our attention.

At a less fundamental level, often there are pleas to simplify the law, to cut "red tape" and paperwork. Indeed, this is incorporated in a provision of the law, in Section 101(f). The classic reply, of course, is in the law itself. That section appears on page 2, but the law continues for another 86 pages of fine print, and almost every page calls for more regulations, guidelines and reports.

Personnel at all levels of the effort in both the public and private sectors have generally been too few. In facilities planning, for instance, no less than ten major categories of requirements must be considered.

Also, there has been the requirement for an equitable system of user charges. The requirement was conceived as an incentive to water and sewer customers to conserve water and cut individual use. Unfortunately, the requirement also placed large administrative and cost burdens on communities with existing ad valorem property-based methods of taxation. On this subject, a California State Representative recently testified that it would cost the Sanitation Districts in Los Angeles County, not including Los Angeles City, over $2 million a year in additional accounting expenses to administer the required system. In short, the user charge concept was environmentally sound, but it was fiscally unsound, in some cases. It failed to take into account established local, revenue-raising practices or the administrative costs of change.

The recovery of the industrial share of treatment construction costs is an allied subject, but since Mr. Taylor will be addressing that area of the law, I will pass over some recent developments there.

So far, I have mentioned only a few of the issues that have "cropped up." As you know, there have been others, and it sometimes seemed last year that the issuance of almost all the required, baseline regulations and guidance by the Agency went largely unnoticed.

But, that was last year, and the year before, This year, I am happy to note that our progress has been rapid, and our momentum is gaining strength. There are several reasons for this and the chief factors, I believe, are the experience we have so painfully gained, and the widespread cooperation we have enjoyed in devising, and implementing the needed changes.

It is also true that "nothing succeeds like success." Recent progress has bred a spirit of optimism in the program. We have been moving to simplify procedures and strengthen management, and where changes are not legally permissible because the law is over specific, we have been promoting legislative amendments.

Concerning the administrative changes, last November, as the result of some in-depth study and program analysis by ourselves and others, we began meeting with representatives of the various interest groups involved in the program--the State and local governments, the consulting engineers, contractors, and wastewater equipment manufacturers.

The recommendations and strong cooperation we have recently received have enabled us to further simplify and streamline the granting procedures and provide guidance in fulfilling them, particularly for those applying to the earliest stages of planning. We have also improved our monitoring capabilities, to project future funding commitments and identify lags and individual projects needing action. We have reorganized for more central management and accountability for the individual projects and the program as a whole, in the Regions and in Headquarters. Since we have received authorization, we are currently recruiting and instituting training for 400 additional positions in the program. But most important, for some time, we have been encouraging the States to assume large parts of the program by assuming larger review and certification responsibilities for project plans and specifications and for reviewing infiltration/inflow and operation and maintenance provisions. We are also helping the States to strengthen their own program activities through increased Section 106 funds, and urging the use of part of the construction grants funds, if needed.

In this latter effort, we have succeeded in encouraging more than half the States to assume extra grant responsibilities. Their management cost increases can currently be covered by withdrawing a sum up to one-half of one percent of their total Federal allotment, and the State of California is taking advantage of this funding option. Alternatively, our EPA budget that is presently before Congress for approval contains extra funding requests to strengthen the States' programming and planning capabilities.

Cumulatively, there has been a good deal of action on the management side this past year. When I consider the total impact of the recent and emerging developments, I am greatly pleased with our progress in the program. In addressing the problem areas, we have introduced a good deal of administrative flexibility.

For the areas we have been unable to address administratively, new legislation has been introduced. A bill to amend PL 92-500, H.R. 9560, is currently awaiting further attention by the House Public Works Committee. Our Administrator, Mr. Train, testified on it at some hearings recently. Briefly, its major grant-impacting provisions are these:

--First, it does contain a provision allowing *ad valorem* taxation to assess user charges. However, we believe it is over-broad and should be limited to only those areas where it has been used historically.

--Second, it does delegate the major construction granting authority to the States. Since we have been working closely with the Public Works Subcommittee on this amendment for some time, and although we do have reservations on the procedure for suspending State certification, we are, for the most part, pretty well satisfied with it.

--Third, it does extend the secondary treatment deadline to July, 1982. On this issue, the Administrator felt the extension should be granted until July, 1983 when Best Practical Treatment would be required. He also indicated the extensions should be on a case-by-case basis, and the non-availability of Federal funding should be a justification for granting an extension. Ocean discharging municipalities, we feel, should not be singled out in the amendment since their discharges can be considered for extension along with the others, if the delay is merited.

Parenthetically, the Task Force on Municipal Ocean Outfalls recommended that ocean dischargers, where the need for secondary treatment is uncertain, should have prime consideration for Step I planning grants to determine their water quality impacts, but should not be given much priority for construction grants, unless indicated by the Step I study.

The remaining provisions of the bill relate to increased funding for State program and areawide planning above what we requested in our Agency budget. As Mr. Train indicated, we disapprove extending areawide planning under Section 208 beyond the initial two years, nor should the 75 percent Federal funding share be increased to 100 percent, as proposed.

Encouragement of strong local participation in planning should be premised on some degree of funding commitment by the localities directly involved. As for the bill's proposal to provide more funds to reimburse advanced construction, we did not support this because we do not think it would buy any additional clean-up of the waters.

Taken all together, and especially if our recommendations are adopted, we feel that the bill will significantly alleviate the short-term problems of the program.

As for the longer-term problems affecting the more distant water quality goals of the law, the National Water Quality Commission's findings, which will be discussed later, when they issue in final form, will be a prime factor next year in determining the final form these goals will assume. Based on the draft version of the staff report that is currently circulating, I am very encouraged by the fact that the municipal treatment costs seem pretty close to our own projections, and the tone seems generally favorable to continuation of the effort.

Looking ahead in the program, I see several major areas of concern. What is particularly needed right now, it seems to me, is more information on the institutional systems, in the localities and particularly the States, that increasingly are becoming involved; on the stresses and strains on these systems; and some assessment of the abilities needed to assume the more substantive parts of the program.

Another concern is with the preservation of adequate operations and maintenance in the plants. As more plants with more sophisticated treatment equipment go on line, operating efficiency will be assuming even greater importance. For these reasons, we have begun actions to strengthen the States' capabilities in operator training and certification programs, also in monitoring.

A third concern is to encourage the development and demonstration of innovative wastewater treatment technology. We are presently in the early stages of developing some possible new legislative initiatives to fill this program need which, hopefully, will be ready for proposal to Congress next year.

Finally, we are concerned with the new State Needs Survey that is just about to get underway. As you may know, we hope to identify from it where we are in terms of reaching our goals, and how far we need to go. In conjunction with this, we will also be scrutinizing closely the State priority lists to be sure the important selection criteria are emphasized. We want to be sure the public funds are not misapplied to projects with marginal benefits to water quality and that the needs that Congress has identified in the law -- for advanced and secondary treatment and for interceptor sewers -- receive first consideration. We want to be sure, in order that we reach our national goals as quickly as possible, that projects emphasized first are those in the critical water quality areas that are required to meet the standards and enforcement provisions of the law.

While on the subject of needs and priorities, I would like to point out that the largest need I foresee in this program is for some assurance of future funding continuity. Municipalities should have some assurance the funds will be there to build and complete their projects before they willingly commit themselves to raise the local funds to pay for planning. Commitments should be made on all sides, and the communities should not have to live under the threat of on-again, off-again Federal assistance.

To learn the views of the public and the professions on how the program should be financed beyond the current authorizations, and how the secondary treatment requirements for municipalities by 1977 or 1978 should be treated when they would be unattainable, EPA held a series of public hearings last June. In summing up the results, as a whole we found a consensus on the necessity to achieve secondary treatment or higher, as required by water quality considerations. Also, we found some support for giving stormwater treatment less emphasis and a lesser priority status, and on extending the 1977/1978 secondary treatment deadline on a case-by-case basis.

After we evaluated these responses and recommendations, we made some estimates based on the cost figures for the 1974 State Needs Survey. As you may know, the total State needs identified in the 1974 Survey, as adjusted, would cost over $342 billion, including the cost of treating and controlling stormwaters, at $235 billion. Since this total sum was just too large to be seriously contemplated, we examined the individual treatment categories in terms of the priorities of the Act, and their relative impact in attaining cleaner water. And we excluded the $235 billion for stormwaters. Since our purpose was to devise a strategy to "optimize" Federal funding, we chose projects to provide secondary and advanced treatment, and interceptor sewers, and to correct sewer infiltration/inflow problems for primary funding emphasis.

And based on our findings and estimates, we expect to make recommendations to the Congress on how the program should be financed in the future. As you may know, we favor a varied Federal funding share depending on relative importance to our clean water goals. And needless to say, this would be premised on continued program funding at a higher level for several more years, following the lapse of the current PL 92-500 authorization.

Parenthetically, I understand Japan is also moving to greatly increase its national funding to municipal treatment projects. I would like to encourage Japan in the significant commitment that will be proposed for national endorsement. The five-year, $30 billion program that is planned, if adopted in its entirety, would constitute a 400 percent increase over Japan's present level of program funding for such construction. I commend your dedication to this significant and laudable water clean-up effort.

In conclusion, I would like to mention some ancillary benefits we expect to derive from the program. Aside from improved water quality -- and since the projects under PL 92-500 are not yet completed, the progress we are seeing is primarily derived from our predecessor law -- one major benefit is to the nation's economy. For instance, employment on the public treatment works is expected to reach a peak of 296,000 new jobs per year in 1977, and remain at over 100,000 jobs through 1983. Overall, in the decade between 1975 and 1985, the National Water Quality Commission has tentatively estimated that implementation of the Act could create a total of nearly one and one-half million man years of work to construct the required treatment works. And the "ripple effect" in the economy will probably create an equal number of additional jobs.

This is a substantial benefit to the nation, but we are not losing sight of our basic objectives -- to clean up our water in the most cost-effective manner possible.

In pursuing this goal, we welcome the opportunity to exchange information, technology and experience with our Japanese counterparts. Although you operate under different laws, our overall goals are similar. I believe these exchanges are valuable to our own water clean-up program, and I wish to take this opportunity to thank the Japanese Government for your significant contributions to improving the water environment of the world.

Thank you.

COMMISSION CHARGE
TENTATIVE STAFF ISSUES AND FINDINGS
NATIONAL COMMISSION ON WATER QUALITY

J. G. Moore, Jr.
Program Director
National Commission on Water Quality
Washington, D. C.

INTRODUCTION

On October 18, 1972, the Congress over a Presidential veto, enacted Public Law 92-500 -- the Federal Water Pollution Control Act Amendments of 1972. Responding to public demand for cleaner water, the law it enacted culminated two years of intense debate, negotiation and compromise, and resulted in the most assertive step in the history of national water pollution control activities.

In their deliberations, Congress determined that the prior program was making too little headway in redressing the serious problems of the Nation's polluted waters. John Blatnik, then Chairman of the House Public Works Committee, testified to the new sense of immediacy of the task at hand -- cleaning up the Nation's waters:

"In this measure, we are totally restructuring the water pollution control program and making a far-reaching national commitment to clean water, much as our space program was restructured a decade ago, when the late President Kennedy committed America to a landing on the moon before the end of the 1960s."

Senator Muskie, bringing the Bill to the Senate floor, stated:

"...today, the rivers of this country serve as little more than sewers to the seas. Wastes from cities and towns, from farms and forests, from mining to manufacturing, foul the streams, poison the estuaries, threaten the life of the ocean depths. The damage to health, the environmental damage, the economic loss can be anywhere."

The Act, P.L. 92-500, departed in several ways from previous water pollution control legislation. It expanded the Federal role in water pollution control, increased the level of Federal funding for construction of publicly owned waste treatment works, elevated planning to a new level of significance, opened new avenues for public participation and created a regulatory mechanism requiring uniform technology-based effluent standards, together with a national permit system for all point source dischargers as the means of enforcement.

In the strategy for implementation, Congress stated requirements for achievement of specific goals and objectives within specified timeframes. The objective of the Act is to "restore and maintain the chemical, physical, and biological integrity of the Nation's waters." In addition, two goals and eight policies are articulated. The goals are:

-- To reach, "wherever attainable", a water quality that "provides for the protection and propagation of fish, shellfish, and wildlife" and "for recreation in and on the water" by July 1, 1983.

-- To eliminate the discharge of pollutants into navigable waters by 1985.

The policies are:

-- To prohibit the discharge of toxic pollutants in toxic amounts.

-- To provide Federal financial assistance for construction of publicly owned treatment works.

-- To develop and implement areawide waste treatment management planning.

-- To mount a major research and demonstration effort in wastewater treatment technology.

-- To recognize, preserve and protect the primary responsibilities and roles of the States to prevent, reduce and eliminate pollution.

-- To insure, where possible, that foreign nations act to prevent, reduce and eliminate pollution in international waters.

-- To provide for, encourage and assist public participation in executing the Act.

-- To pursue procedures that drastically diminish paperwork and interagency decision procedures and prevent needless duplication and unnecessary delays at all levels of government.

The Act provides for achieving its goals and objectives in phases, with accompanying requirements and deadlines.

Phase I, an extension of the program embodied in many State laws and Federal regulations, requires industry to install "best practicable control technology currently available" (BPT); and publicly owned treatment works to achieve secondary treatment -- by July 1, 1977 -- as well as "any more stringent limitations, including those to meet (State or Federal) water quality standards." (Sec. 301(b)(1)(C)).

Phase II requirements are intended to be more rigorous and more innovative. Industries are to install "best available technology economically achievable (BAT)... which will result in reasonable further progress toward the national goal of eliminating the discharge of all pollutants"; and publicly owned treatment works are to achieve "best practicable waste treatment technology... including reclaiming and recycling of water, and confined disposal of pollutants" (BPWTT) -- by July 1, 1983 -- as well as any water quality related effluent limitation. (Section 302) Ultimately, all point source controls are directed toward achieving the national goal of the elimination of the discharge of pollutants by 1985.

The Act was intended to be more than a mandate for point source discharge control. It embodied an entirely new approach to the traditional way Americans have used -- and abused -- their water resources. Some of these mechanisms are found in Title I, the broad policy title; others are woven through-out the Act in grants and planning, in standards and enforcement and in permits.

The second section of the Act requires the development of comprehensive programs for preventing, reducing and eliminating pollution, and further asks for research and development aimed at eliminating unnecessary water use. In Section 208, the statute directs the designation of areawide institutions to plan, control and maintain water quality and reduce pollution from all sources through land use or other methods.

Construction grants for publicly owned treatment works are made available to encourage full waste treatment management, providing for:

"(1) the recycling of potential sewage pollutants through the production of agriculture, silviculture, and aquaculture products, or any combination thereof;

"(2) the confined and contained disposal of pollutants not recycled;

"(3) the reclamation of wastewater; and

"(4) the ultimate disposal of sludge in a manner that will not result in environmental hazards."

The grantees are encouraged to combine with other facilities and utilize each other's processes and wastes. Facilities are to be designed and operated to produce revenues.

These statutory provisions outline a long-term program to reduce water use, reduce the generation of wastes and establish financially self-sustaining publicly owned pollution control facilities.

Congress recognized it was placing demands on people, technology, the economy and public and private institutions which, ultimately, would determine the success of the program. There was uncertainty as to whether the required technologies existed; how the new law's implementation might affect society, the economy, and the environment; and how well the institutional system would or could respond to this demanding new law.

The Commission was established by Section 315, to determine if a mid-course correction was needed between 1972 and 1983, and to report its findings, conclusions and recommendations to the Congress within three years from the law's enactment.

COMMISSION CHARGE

The Commission's charge is:

To "...make a full and complete investigation and study of all the technological aspects of achieving, and all aspects of the total economic, social, and environmental effects of achieving or not achieving, the effluent limitations and goals set forth for 1983 in Section 301(b)(2) of this Act."

The Commission determined "that a comprehensive study of the goals and requirements for 1983 cannot be properly undertaken without attention to the progress made toward clean water by industries and municipalities under the 1977 requirements." It stated that it would "examine progress toward the 'elimination of the discharge of pollutants' as an indicator of what will remain to be done after 1983."

Because of the specific statutory charge, the goal and effluent limitations for 1983, and the impacts of their application have remained the primary focus of the Commission's attention. In the time permitted, no study could assess all the far-reaching implications of each of the Act's provisions. Limitations of data, time and resources, as well as evolving knowledge and issues, restrict what can be done and still meet deadlines. The Commission has completed a major undertaking and has produced new and useful information for the Congress. The Commission concludes its assignment with the knowledge it has both answered and generated questions. Its report contains the following components:

Chapter I -- An analysis of the capabilities and costs of technology for achieving the effluent limitations required by 1977 and 1983, as well as a look at what remains to be done after 1983;

Chapter II -- An assessment of the capability of the public and private sectors to apply the defined effluent limitations for 1977 and 1983, as well as those more stringent to protect water quality;

Chapter III -- An analysis of the economic effect of the cost of applying the necessary technologies on both a macroeconomic and microeconomic scale -- as well as the social effects of these changes;

Chapter IV -- A description of present water quality and environmental (water-based and related terrestrial) conditions, and projections of anticipated change which may result from implementation of the Act.

Chapter V -- Identification of the effects in several selected regions of the Nation; and

Chapter VI -- An assessment of the public and private response as the institutional segments finance, implement, manage and enforce the Nation's water pollution control program.

[The discussion which follows presents the Commission staff's tentative summary of the issues and findings. Recommendations will be formulated by the Commission as it completes its deliberations and concludes its work.]

CAVEAT

Commission studies performed by contractors often provide a range of costs and benefits for differing technologies, conditions, or assumptions. Complete and reliable data were not always available covering all questions the Commission and contractors have analyzed; every effort is made to explain methodologies and to qualify estimates in the full text of the Commission staff draft. In the following summary, reflecting this variability has been sacrificed for the sake of brevity. Therefore all cost and expenditure estimates and quantified statements of effects should be regarded as staff best professional judgment rather than as exact measures of these items. The staff recognizies that others might have chosen other figures for different reasons.

CAVEAT

NOTE: All dollar figures are stated in June 1975 dollars.

I. Progress Under 1977 Requirements

Question: How will progress toward achieving the 1977 requirements affect the timetables for "achieving or not achieving the effluent limitations and goals set forth for 1983"?

Answer: The effect is to delay achievement of 1983 requirements and goals. Slow progress toward achieving secondary treatment by publicly owned treatment works by July 1, 1977, will delay achievement of "best practicable waste treatment technology over the life of the works" by 1983. Similarly, but to a lesser degree, industries and agriculture not achieving "best practicable control technology currently available" by July 1, 1977, will delay achievement of "best available technology economically achievable" by July 1, 1983.

A. Technological Aspects - 1977

Publicly Owned Treatment Works - 1977

Question: Is secondary treatment for publicly owned treatment works as defined by the Administrator of the Environmental Protection Agency technologically achievable?

Answer: Yes.

Question: Will all publicly owned treatment works achieve secondary treatment by July 1, 1977?

Answer: No.

Question: When might all publicly owned treatment works achieve secondary treatment?

Answer: Achievement is dependent primarily on Federal funding.* For example, it could be achieved in 11 years with 75 percent Federal funding of all eligible construction categories except control and treatment of separate storm sewer flows with a total Federal outlay of $118.5 billion, provided Federal appropriations and commitments are made ranging from $2.6 billion the first year to $15.6 billion in the last two of the eleven years. This pattern would require radically accelerated processing of all phases of grant administration and treatment plant construction. (Inflation could increase the total to $184.4 billion.) Achievement of some eligible categories could be realized earlier depending upon selective Federal funding.

Industries - 1977

Question: Is "best practicable control technology currently available" as defined for industrial dischargers technologically achievable?

Answer: Generally, yes, with the exception of short term limits in some cases and waste constituents or mixed wastewater streams in others.

Question: Will all industrial dischargers achieve best practicable control technology currently available by July 1, 1977?

Answer: No.

Question: When might all industrial dischargers achieve best practicable control technology currently available?

Answer: Perhaps by 1980; industries appear to be moving to achieve the 1977 requirement at a rate that will assure achievement earlier than publicly owned treatment works will achieve their 1977 requirements.

---

*H.R. 9560, Sec. 9, would allow time extension on a case by-case basis but in no event later than July 1, 1982.

Capital Costs for 1977

Technology Costs	Amount (Billions, 1975 $)
Iron & steel	$2.91
Organic & miscellaneous chemicals	4.29
Inorganic chemicals	0.52
Petroleum refining	1.05
Pulp & paper	2.64
Metal finishing	14.14
Fruits and vegetables	0.44
Plastics & synthetics	0.16
Textiles	0.54
Steam electric power*	3.74*
Feedlots**	2.21**
All other industry***	11.67***
TOTAL	$44.31

---

*Includes growth to 1977.

**Includes growth to 1977 and assumes coverage of all feedlots. Estimated costs for EPA's most recent proposal (November 1975) would be $0.11 billion.

***See Table p. 10 for listing.

Capital Expenditures for 1977*

Economic Impact Expenditures*	Amount (Billions, 1975)
Iron & steel	$2.08
Organic and miscellaneous chemicals**	4.29**
Inorganic chemicals	0.81
Petroleum refining	0.83
Pulp & paper	2.33
Metal finishing	9.13
Fruits and vegetables	0.16
Plastics and synthetics	0.21
Textiles**	0.54**
Steam electric power***	4.09***
Feedlots****	0.80
All other industry	11.28
TOTAL	$36.5

---

*For plants in place as of June 1973; no growth from that date is included.

**Does not have adjustments for closures.

***Includes all costs for existing plants not entitled to exemption to meet thermal requirements which become effective July 1, 1983.

****Includes all categories of feedlots with a limited number covered in each category.

B. Economic Effects - 1977

Publicly Owned Treatment Works

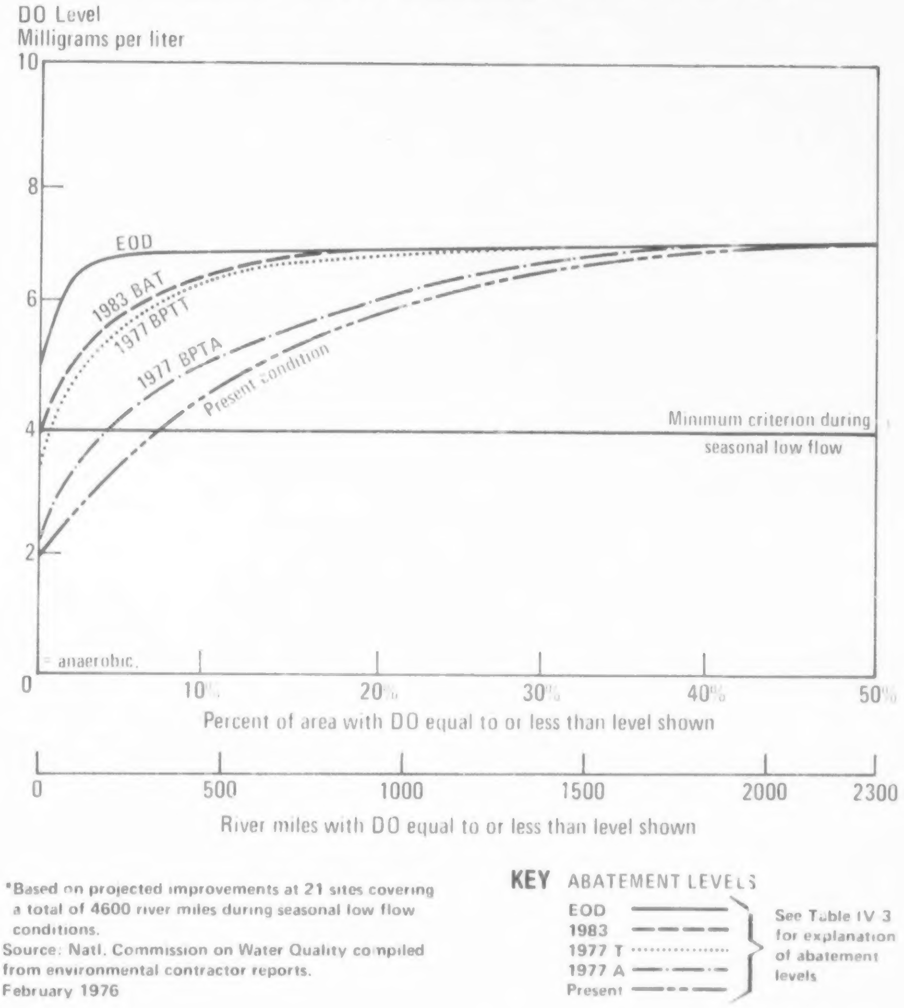
Categories	Minimum	Amount (Billions, 1975 $)	Maximum	Calendar Year Achieved
I. Secondary treatment		10.8		1980
II. Treatment more stringent than secondary		24.8		1980
IIIA Correction (infiltration/inflow)		6.9		1985
IIIB Sewer rehabilitation		9.5		1985
IVA Collector sewers		13.0		1985
IVB Interceptor sewers		13.5		1980
V. Combined sewers	$5.4	79.6	88.4	1985
SUBTOTAL	$83.9	$158.1	$166.9	
VI. Control of storm-waters	158.0	199.0	427.0	
TOTAL	$241.9	$357.1	$593.9	

C. Environmental Effects - 1977

Question: How far will the uniform application of BPT to industrial dischargers and secondary treatment to publicly owned treatment works go toward achievement of the interim goal, a water quality "wherever attainable" "which provides for the protection and propagation of fish, shellfish, and wildlife and provides for recreation in and on the water . . . by July 1, 1983"?

Answer: Based on the analysis of the Commission's 41 environmental sites, application of the 1977 requirements will restore a large portion of the nation's presently polluted waters to a level of physical and chemical quality sufficient to provide for achievement of the interim goal. The chief exceptions are caused by toxics, pulse loads discharged from point and nonpoint sources during and following storms and delays in actual achievement of the 1977 requirements. Maintaining that level of quality with continued growth will depend upon timely and effective compliance with outstanding permits and the effective application of more stringent limitations that may become necessary following adoption of water quality standards where the volume of pollutant discharges begins to produce lower water quality.

DISSOLVED OXYGEN IMPROVEMENT
UNDER 4 LEVELS OF POLLUTION CONTROL ABATEMENT



*Based on projected improvements at 21 sites covering a total of 4600 river miles during seasonal low flow conditions.
Source: Natl. Commission on Water Quality compiled from environmental contractor reports.
February 1976

II. "Technological aspects of achieving"; "economic, social and environmental effects of achieving or not achieving, the effluent limitations and goals set forth for 1983. . . ."

A. Technological Aspects - 1983

Publicly Owned Treatment Works - 1983

Question: Does technology adequate to achieve "best practicable waste treatment technology over the life of the works" for publicly owned treatment works exist?

Answer: Yes, since as defined by EPA this technology is virtually the same as that described by EPA for 1977.

Question: Will all publicly owned treatment works achieve the "best practicable waste treatment technology over the life of the works" by July 1983?

Answer: No. The primary reasons for not achieving this requirement by the scheduled date is an inadequate rate of Federal funding, slow obligations of existing funds and the time it takes for construction of treatment works.

Question: When might all publicly owned treatment works achieve best practicable waste treatment technology over the life of the works?

Answer: Depends primarily on level of Federal funding. Could be achieved in eleven years, so long as there is little technological difference between the 1977 and 1983 requirements and an adequate level of Federal funding, commitment of the funds and construction of plants is maintained.

Industries - 1983

Question: Is defined "best available technology economically achievable" for industrial dischargers technologically attainable?

Answer: Generally, yes, with the exception of short term (24-hour) limitations in some cases and those instances where application of technologies must be transferred from one industry to another or have not been adequately demonstrated.

Question: Can industrial dischargers attain best available technology economically achievable by July 1, 1983?

Answer: Depends upon date the 1977 requirements are actually met and resolution of challenges to some effluent limitations and permits.

Question: Should the deadline for industry to achieve "best available technology economically achievable" remain July 1, 1983?

Answer: No, and a "mid-course correction" appears indicated.

Added Costs for 1983 Requirements

Technology Costs	Amount (Billions 1975 $)	
Iron & steel	$0.95	
Organic & misc. chemicals	3.64	
Inorganic Chemicals	0.25	
Petroleum refining	1.18	
Pulp & paper	0.80	
Metal finishing	14.09	
Fruits & vegetables	0.16	
Plastics & synthetics	0.29	
Textiles	0.30	Maximum
Steam electric power*	2.03*	$7.96
Feedlots**	0.49**	
All other industry	6.38	
TOTAL	$30.56	$36.49

---

*Includes growth to 1983. Range depends on assumed exemptions.

**Includes growth to 1983. Estimated costs for EPA's most recent proposal (November, 1975) would be $0.04 billion.

B. Economic Effects - 1983

Added Expenditures for 1983 Requirements

Economic Impact Expenditures	Existing Plants Amount (Billions 1974 $)	New Sources 1973 to 1983 Amount (Billions 1975 $)
Iron & steel	$ .55	$ .65
Organic & miscellaneous chemicals	3.64*	2.25
Inorganic chemicals	.26	.35
Petroleum refining	1.18*	.29
Pulp & paper	.76	.73
Metal finishing	8.25	3.97**
Fruits & vegetables	.11	.05
Plastics & synthetics	.29*	.09
Textiles	.30*	.28
Steam electric power	1.28	.99
Feedlots	.22***	.23
All other industry	6.17	10.11
TOTAL	$23.01	$19.90

---

*Does not have adjustment for closures.

**Excludes captive shops included in machinery and mechanical products, $8.30 billion, under all other industry.

***Includes all categories of feedlots with a limited number covered in each category.

Table III.P.L

INDUSTRY

Preliminary Economic Impact Expenditure Estimate
(Millions of 1975 dollars)

	BPT (1977)		BAT (1983)		1975-83 NSPS
Indepth	Capital	Annual O&M	Capital	Annual O&M	Capital
Fruits & Vegetables	155	14	111	12	47
Inorganic	805	178	261	140	351
Organic	3328	487	2990	2242	1873
Misc. Chemicals	965	163	653	228	380
Iron and Steel	2080	261	546	203	647
Metal Finishing-Job	1715	328	780	168	3967
Captive	7418	2275	7468	1365	a
Petrol. Refining	829	142	1184	429	294
Plastics & Synthetics	209	33	286	29	91
Pulp and Paper	2331	117	757	36	734
Steam Electric	4089	989	1275	16	900
Textiles	537	88	300	29	275
	24461	5075	16611	4897	9559
Other					
Ore Mining & Dressing	611	25	0	0	146
Coal Mining	1690	95	0	0	0
Petrol. & Gas Ext.	234	18	1069	61	266
Mineral Mining & Proc.	728	72	0	0	496
Meat Products & Rendering	148	20	179	16	64
Dairy Products	198	14	66	5	0
Grain Mills	56	4	9	1	0
Cane Sugar Processing	153	17	165	13	8
Beet Sugar	90	17	69	5	2
Canned & Preserved Seafood	55	14	105	5	38
Misc. Food & Beverages	--	0	5	1	4
Timber Products	14	1	25	8	33
Furniture & Fixtures	7	3	0	0	2
Bldg. Paper & Board	120	12	0	0	13
Paint & Ink	23	23	0	0	2
Soap & Detergent	8	1	1	1	2
Phosphate Mfg.	108	13	26	3	46
Fertilizer Mfg.	73	47	68	22	50
Paving & Roofing	7	7	4	1	1
Rubber Processing	221	18	48	12	299
Leather Tanning	77	20	39	8	13
Glass Mfg.	35	4	57	9	26
Cement Mfg.	34	4	9	1	12
Concrete, Gypsum, Plaster	100	26	0	0	36
Asbestos	4	1	9	4	3
Insult. Fiber	13	5	0	0	3
Ferroalloy Mfg.	48	16	10	4	11
Nonferrous Metals	40	21	31	7	39
*Machinery & Mech. Prod.	3900	293	3900	293	8295
Transportation Ind.	866	88	143	39	137
Water Supply	1222	156	104	3	28
Auto & Other Laundries	18	3	21	1	8
Foundries	182	26	0	0	0

(continued)

Table III.P.L (continued

INDUSTRY

Preliminary Economic Impact Expenditure Estimate
(Millions of 1975 dollars)

	BPT (1977)		BAT (1983)		1975-83 NSPS
Other	Capital	Annual O&M	Capital	Annual O&M	Capital
Fish Hatcheries	--	--	--	--	--
Structural Clay	5	1	--	--	2
Pottery	3	1	4	3	1
Steam Supply	0	0	0	0	--
Nonferrous Mills	195	18	--	--	33
Feedlots:					
Beef	185	13	42	0	225[b]
Hog	445	22	117	0	--
Dairy	265	20	56	0	--
All Other Industries	12181	1159	6381	526	10344
In-depth Industries	24461	5075	16611	4897	9559
TOTAL	36642	6234	22992	5423	19903

*Excludes Captive Metal Finishing

a Included in Machinery and Mechanical Products

b Includes all categories of feedlots with a limited number covered in each category.

National Commission on Water Quality
October 2, 1975

C. Social Effects and Benefits - 1977 and 1983

Question: Who is likely to bear the adverse social impacts from the achievement of the water quality requirements and goals?

Answer: Adverse impacts will fall most heavily on the employees and owners (either individual or corporate) of those industrial plants closed as a result of the implementation of the 1977 and 1983 effluent limitations. Generally, small older businesses will be adversely affected. Employment losses from plant closures, both direct and indirect, could amount to over one-half million jobs with full implementation of BPT and BAT limitations. Other categories adversely impacted will be the moderate-income family seeking to enter the housing market for the first time, the low-income resident who will bear a higher relative cost of the local share of financing pollution control facilities relative to his income as a result of the shift from property taxes to user fees.

Question: How many jobs will be created by the implementation of P.L. 92-500? What kind of jobs will they be and how well distributed geographically and demographically?

Answer: The major direct labor requirement will be for construction workers to build the plants and facilities. Heaviest impacts will be in the area of publicly owned treatment systems and sewers, where employment will reach a peak in 1977 of 296,000 new jobs and remain at over 100,000 through 1983*. Overall, between 1975 and 1985, implementation of the Act will create a total of nearly one and a half million man-years of work in the construction of publicly owned waste treatment facilities.

Question: Who benefits? Are there identifiable publics that receive the benefits derived from improved water quality?

Answer: Sports fishermen, commercial fishermen, seafood canners and processors and those commercially associated with water-based recreational pursuits -- especially recreational boating -- will be major beneficiaries of the Act. Property owners, too, of riverside and shoreline properties will enjoy not only the benefits of improved water quality adjacent to their property, but also the increased value of the property as well.

Question: Are the benefits from the achievement of the 1983 interim water quality goal quantifiable?

Answer: Yes. Improved water quality will provide the American public with greatly expanded opportunities for water-based recreation, sports fishing and the commercial harvesting of fish and shellfish. Each of these activities will result in directly measurable benefits to individuals, segments of society and to the economy in general. There will be additional unquantifiable benefits resulting from aesthetic changes in water bodies and water-related activities.

Question: Economically, what will these benefits amount to?

Answer: All quantified benefits, translated into dollar gains, may increase from an estimated annual rate of $3.4 billion in 1980, to $5.2 billion in 1985, and $7.8 billion in the year 2000, assuming gradual improvement from the present to 1985 with increases in benefits thereafter resulting solely from population and economic growth.

Question: Are these short-term benefits?

Answer: No. Assuming the maintenance of the water quality, these gains will continue to accrue into the future. For swimming alone, the cumulative benefits from 1985 to the year 2000 could reach between $16 and $17 billion, and annual quantifiable benefits for all activities could reach $6.4 billion by the year 2000.

Question: Are there unquantifiable benefit associated with the achievement of the 1983 interim water quality goal?

---

*Assumes BPT met by 1977 with expenditures over three years, and BAT met by 1983 with expenditures spread over six years; and $60.9 billion for publicly owned treatment works spread over eleven years.

Answer: Yes, two important kinds: (1) those not fully identified yet, such as health effects, where quantification now is impossible; and (2) those associated with non-contact related experiences.

II. D. Environmental Effects - 1983

Question: Do incremental improvements in water quality characteristics associated with achievement of the 1983 effluent limitations contribute significantly to the realization of the 1983 goal?

Answer: In most cases, the gains are projected to be less than those indicated for achievement of the 1977 requirements.

III. Technological Aspects and Economic Effects of 1977 and 1983 Requirements Applied to Agriculture

Agriculture - 1977 and 1983

Question: Are agricultural point sources of pollutants amenable to the technological concepts of BPT and BAT?

Answer: Some are, such as feedlots. Others, such as irrigation return flows, present unique and widely variable problems when attempting to apply effluent limitations requirements of P.L. 92-500.

Question: What is being done to implement BPT for irrigation return flows?

Answer: Since EPA has defined the 1977 requirement as monitoring the quantity and quality of intake water supply and irrigation return flows, BPT as so far defined, is technologically achievable. Complex and widely varying systems by which excess water applied for irrigation reaches surface and ground waters in the vicinity of its application, do present even monitoring problems, however. Known control technologies currently available are not so well developed as to assure their universal practicable application with predictable results.

Question: Is "best available technology economically achievable" to control pollutants in irrigation return flows technologically attainable by July 1, 1983?

Answer: No, simply because no universally applicable or effective technology has been developed which can be applied with reasonably predictable results for all geographic areas.

Question: Should statutory provisions intended to control pollution from irrigation return flows remain a part of P.L. 92-500?

Answer: Yes, but they should be so formulated and designed as to recognize the variety of polluting effects, the unique institutional structure of irrigated agriculture, the wide geographical differences in irrigation objectives and practices and the state-of-the-art for control practices and measures for reducing pollutants from irrigated agriculture.

IV. Institutional Factors Influencing "Technological Aspects of Achieving" and "Economic, Social, and Environmental Effects of Achieving or Not Achieving, the Effluent Limitations and Goals Set Forth for 1983....."

Institutional Factors - 1977 and 1983

Question: How has the response of the national institutional structure for water pollution control -- Federal, State, interstate, regional and local governments, industrial and agricultural dischargers and all interested groups -- affected achieving or not achieving by the stated deadlines all that is required to meet the requirements and goals of P.L. 92-500?

Answer: Individually and collectively, their response has contributed to the time required to achieve results under P.L. 92-500. Unfortunately, the interrelationships have too often been adversary rather than cooperative in nature.

Question: What are the implications of the resultant delays in meeting deadlines or implementing various statutory directives?

Answer: The obvious implication is that the time schedule for accomplishment of some requirements and goals of the Act is out of phase. The first major deadline -- installation by dischargers of Phase I wastewater treatment technologies by July 1, 1977 -- will not be met by all dischargers, and the series of delays which have occurred within the institutional structure and the slow flow of grant funds share major responsibility. This fact requires consideration of alteration in the present time schedule for accomplishment to coincide more closely with the realities of current circumstances and available funding as well as the limitations inherent in the intergovernmental process and the interactions between these governments and the various affected and participating "publics".

Question: Where in the program structure are the delays most seriously manifested, and what do they reveal about the fundamental implementation strategy of the Act?

Answer: The strategy of the Act is predicated on the proposition that 75 percent Federal construction grant assistance for publicly owned treatment works, coupled with strong centralized national initiatives for regulation and enforcement structured within a framework of comprehensive state and local planning, will produce the most expeditious, effective and sensitive application of resources and manpower to the accomplishment of national water quality objectives. In the first three years a divergence has evolved between the proposition and its realization. Delays in issuance of guidelines, effluent limitations, and regulations, delays in the obligation and outlay of Federal funds for construction grants, delays and variations in the issuance of the NPDES permits for municipal, industrial and agricultural dischargers and the fact that mandated planning requirements are seriously out of synchronization with the construction grants and permit phases, all contribute to uncertainty as to the validity of the essential proposition. Experience with implementation to date can neither effectively discredit nor irrefutably sustain the proposition. Experience can, however, identify the points in the implementation process where the delays have been most pronounced and make some observations, based on performance thus far, about the basic structure of institutional cooperation vital to the Act's fair and effective implementation.

A. Regulation

Question: Has implementation progressed sufficiently to identify any basic problems with the regulatory strategy contemplated in P.L. 92-500?

Answer: While there are still a number of uncertainties about the implementation of the regulatory scheme, the conduct of two activities in the regulatory strategy can jeopardize the effective implementation of the Act. One is the development and promulgation of effluent limitations which are being aggressively contested by industry in over 250 legal challenges. The other is the capability of the Federal, state and local regulatory and administrative agencies to develop useful information systems for monitoring and reporting compliance with the Act's various program components.

B. Financing

Question: Has the flow of Federal construction grant funds to the states for publicly owned treatment works impacted the timely achievement of the 1983 requirements and the interim goal?

Answer: Yes.

C. Planning

Question: Has implementation of the planning provisions of the Act progressed commensurate with the planning policy objectives articulated in Sec. 101(a)(5) -- "that areawide waste treatment management planning processes be developed and implemented to assure adequate control of sources of pollutants in each state"?

Answer: No. The planning process is already out of sequence with the implementation strategy.

D. Public Participation

Question: Has the experience with public participation in water pollution control programs differed significantly under P.L. 92-500 from prior experience?

Answer: No. Even though the Act mandated an unprecedented public role, there is no empirical evidence to suggest that the actual influence of the citizen on the decision-making process has effectively increased.

V. What Will Remain to be Done After 1983

Elimination of the Discharge of Pollutants - the 1985 Goal

Question: Assuming achievement of the 1977 and 1983 requirements for publicly owned treatment works and agricultural and industrial dischargers at some future date later than 1983, what major pollutant sources will continue to prevent realization of the interim and ultimate goals?

Answer: Any point source (such as urban runoff in separate storm sewers) not adequately controlled with the 1977 and 1983 requirements or any nonpoint source (such as urban runoff from other than combined sanitary and storm and separate storm sewers and agricultural or general land runoff) contributing pollutants to the nation's waters can prevent achievement of the water quality goals of the Act. The effect of these source contributions in some site specific areas of the country will prevent achievement of the water quality goals of the Act and may, in some cases, overwhelm improvements from point source control. To the extent that publicly owned treatment works, industrial and agricultural dischargers have not achieved elimination of the discharge of pollutants, they, too, would be major contributors.

Question: Are there technologies available now, or that might reasonably be expected to become available within the next 10 years, that could achieve the elimination of "the discharge of pollutants into the navigable waters"?

Answer: Yes. However, in most cases the costs of these technologies are such that, however the technologies might be applied to point source discharges, their installation would be prohibitively expensive, and the economic and social effect would be too severe to be absorbed within the foreseeable future; further, predictable environmental effects, using present analytical techniques and methodologies, would appear to be minimal, particularly in the absence of adequate control of nonpoint sources of pollutants in some places.

Question: Does the goal of the elimination of "the discharge of pollutants into the navigable waters" have value as a guide to policymakers?

Answer: Yes. Wastes generated by projected population increases and continued economic growth, if discharged to the nation's waters, can soon minimize the effect of improvements in receiving water quality realized through achievement of the 1977 and 1983 requirements of the Act applied to point source discharges. If new sources of pollutants -- new industrial plants, agricultural activities and increased discharges from publicly owned treatment works -- install "the best available demonstrated control technology, processes, operating methods or other alternatives, including, where practicable, a standard permitting no discharge of pollutants" [Sec 306(a)(1)], and if existing water quality standards are regularly reviewed and appropriate waste load allocations timely made, this action may assure that growth does not negate the improvements achieved by point source effluent limitations as the program envisioned by P.L. 92-500 is implemented.

Question: Is progress being made toward the "objective of this act . . . to restore and maintain the chemical, physical, and biological integrity of the nation's waters"?

Answer: Yes, although progress to date attributable to P.L. 92-500 is minimal, since results from its implementation are only now beginning to be realized.

Question: Can this "objective" serve as a guide to policy-makers for their actions?

Answer: Not with much certainty or specificity. Even the experts cannot agree upon a full statement of the objective's meaning. "Restore" obviously means returning to some prior condition; restoring "physical" integrity could mean eliminating man-made changes in the nation's waterways so that sediment loads and temperatures would be as they were before man's perturbations of the land; restoring "chemical" and "biological" integrity would require knowing these conditions of the nation's waters at some historical point in time with a degree of accuracy not possible from existing historical data. Man has consistently adjusted, or attempted to alter, the "biology" of the nation's waters to suit his perceived needs. These adjustments or alterations would have to be eliminated in any literal application of the objective -- often to man's discomfort, if not to the detriment of his well-being or health.

PB-265 634

ERRATA

"UPDATED STATUS OF OXYGEN-ACTIVATED SLUDGE WASTEWATER TREATMENT"
Author: R. C. Brenner
PROCEEDINGS 4th US/Japan Conference on Sewage Treatment Technology - 1975

---

page 512 - right hand column - 1st paragraph - line 14

Starting with - The oxygen dissolution...

Change to: Oxygen dissolution capability per diffuser is rated at 95 percent at an oxygen flow of 1500 lb/day (680 kg/day) in wastewater for a total system dissolution capacity at Denver of 9.25 tons/day (8.4 metric tons/day). Previous proprietary tests indicated these diffusers can be operated at oxygen flows up to 33 percent over their rated capacity with only a 5 percent decrease in oxygen transfer efficiency. On this basis, assuming an average BOD_5 removal of 140 mg/l and an oxygen requirement of 1.3 lb O_2/lb BOD_5 removed (1.3 kg/kg), the maximum sustained wastewater flow which can be handled by Denver's 13 RAD's is 15.4 mgd (0.67 cu m/sec).

* * * * * * * * *

page 514 - right hand column - 2nd paragraph

Under Phase I 1976

Change @ 2 mgd to: @ 9 mgd

Under Phase III 1976

Change @ 2 mgd (0.34 cu m/sec) to: @ 9 mgd (0.39 cu m/sec).

* * * * * * * * *

page 515 - left hand column - 1st paragraph - line 7

Change (0.4 cu m/sec) to: (0.39 cu m/sec).